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Aquatic weed control within an integrated water management framework

E.P. Querner

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Abstract

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A combined surface and groundwater flow model for analysing the effects of weed control was developed. The model was verified with field data on groundwater levels, discharges, moisture storage in the root zone, and evapotranspiration in the Hupselse Beek catchment. Weed control strategies were evaluated for a pilot area in the eastern part of the Netherlands. In a cost-benefit analysis the cost of weed control and the damages for pasture were considered. The multi-objective decision method ELECTRE II enabled ecological impact to incorporated as well. The method identified weed control strategies that are most favourable for agriculture or nature conservation.

Keywords: cost-benefit analysis, drainage, ecology, flow resistance, groundwater flow, surface water, unsaturated zone, weed obstruction, weed control.

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Preface

The work presented in this report was initiated in the period 1987-1988 when the author was a member of the working group on Aquatic weed control of the Union of Water Boards in the Netherlands. The research work was started in 1989 in close cooperation with the water board Regge and Dinkel.

For the Hupselse Beek catchment meteorological and hydrological data was obtained from Rijkswaterstaat RIZA. Data on the surface water system was obtained from the water board de Berkel.

Together with the water board Regge and Dinkel groundwater levels and discharges were measured in the pilot area Poelsbeek and Bolscherbeek. The data of the surface water system of these catchments and the information on weed control carried out by the water board were supplied.

The author gratefully acknowledge the support and critical remarks of dr. P.J.T. van Bakel (former SC-DLO; presently TAUW Infra Consult) and ir. J.H.A.M. Steenvoorden (SC-DLO). Mr. H. Breunissen (SC-DLO) carried out experiments in a laboratory flume and assisted with the field measurements. Ing. P. Droogers, ing. C.C.P. van Mourik (both from Int. Agricultural College Larenstein) and ir. J. Babos (University of Budapest) carried out the initial calculations with the MOGROW model. Together with ir. R.H. Pitlo (Advisory Group on Vegetation Management in Wageningen) velocity measurements were carried out in two water courses in Salland.

The work presented in this report has been used as doctoral thesis for the University of Wageningen. The promoters for this work were dr. J.J. Bogardi (Professor in hydrology, hydraulics and quantitative water management) and dr. ir. R.A. Feddes (Professor in soil physics, vadose zone hydrology and groundwater management). Their positive critism and comments on the draft chapters of this report contributed to a large extend to the improvement of this report.

Erik Querner

Summary

In this research the weed control carried out by the water boards in the Netherlands was examined, using hydrological models and weed growth characteristics. Weed control is necessary to maintain an acceptable transport capacity in water courses during the growing season. This means that weed control influences the levels of surface water and of groundwater. A hydrological model to analyse weed control must therefore include surface and groundwater flow. To achieve this, the groundwater flow model SIMGRO was combined with the surface water model SIMWAT. This integrated surface and groundwater flow model, MOGROW, was verified in the Hupselse Beek area. Flow resistance and weed growth also had to be considered in this research. They were studied in the pilot areas of Poelsbeek (41 km^2) and Bolscherbeek (23 km^2) in the eastern part of the Netherlands near Enschede. Different weed control strategies were simulated numerically. The results were evaluated by means of a cost-benefit analysis and a multi-objective decision analysis. The main research topics are discussed below.

Combined surface and groundwater flow model MOGROW

The model MOGROW was developed to simulate the flow of water in the saturated zone, the unsaturated zone and the surface water. The model is physically-based and can therefore be used in situations with changing hydrological conditions. The saturated part consists of a quasi three-dimensional finite element model. The unsaturated zone is modelled one-dimensionally per land use. Special processes were included in the unsaturated zone model, they were: surface runoff, perched water tables, hysteresis and preferential flow. The major water courses are considered in the model as a network of sections. This network can also include special sections such as weirs, pumps, gates and culverts. The minor channels are considered as aggregated reservoirs connected to the network.

The MOGROW model was verified on the Hupselse Beek catchment. It is relatively simple to simulate the variation in groundwater levels, but it is more difficult to represent peak discharges for such a small catchment. Incorporating the process of preferential flow and surface runoff in the model of the unsaturated zone gave more realistic discharges.

Flow resistance

The roughness coefficient required in the Manning formula was calculated from field data and laboratory experiments. A good solution appeared to be to use the unobstructed part of the cross-section only and exclude the part covered by weeds.

The obstructed part has a very low flow rate and can be ignored. The roughness coefficient $k_M°$ for the unobstructed part varied between 30–34 $m^{1/3} \cdot s^{-1}$.

Using a numerical approach to estimate velocity distribution
The SIMVEL model was used to estimate the velocity distribution in a cross-section of a water course with weed growth. It estimates the flow rate from the calculated velocity distribution and then derives a roughness coefficient. These roughness coefficients were in the same order as computed directly from the field data using the Manning formula.

The SIMVEL model was also used to estimate the flow rate of a water course, using some field data. The model then substitutes some of the recorded velocities in the field and calculates the velocity for those parts of the cross-section where no measurements have been carried out. The resulting velocity distribution over the entire cross-section is used to calculate the flow rate. For a water course of approximately 6.5 m wide on the water surface, only 18 point measurements gave a good estimate of the flow rate (50 measurements were used originally).

Weed obstruction
Weed obstruction was measured throughout the growing season. The data were obtained from water courses without any weed control and also in sections where weeds were cleared twice during the growing season. The relative obstruction during the growing season of 1990 and 1991 remained about the same. The data were presented in three classes expressing the average water depth (Figs. 62 and 63).

Scheduling weed control using expected discharges and permissible flow rates
The weed control necessary for a water course can be estimated from the discharge characteristics and data on weed growth during the growing season. The discharge characteristics were calculated using a one-dimensional model of the unsaturated zone. The result of 40-year simulations is the variation in discharges over the summer period. The permissible flow rate of a water course was calculated for the unobstructed part only, using a roughness coefficient $k_M°$. Based on the variation of the expected discharge and the permissible flow rate over the growing season, the required periods of weed control were estimated. The weed control required in two water courses in the Poelsbeek catchment was estimated, using three water table fluctuation classes (Gt III, IV and VII) to give an estimate of the discharge characteristics. An emergency overflow from the sewer system of the urban area was also incorporated. Both locations require one clearing during the growing season, but the actual timing is different (Table 31).

Approach used to evaluate weed control

The method of evaluation was tested in a pilot area comprising the Poelsbeek and Bolscherbeek catchments. The land use is predominantly pasture, silage maize and woodland. The MOGROW model was used to carry out numerical experiments to evaluate different weed control strategies. The results (incidence of high surface water levels, high groundwater levels and high stream flow velocities) were recorded. These numerical experiments were evaluated first by comparing the cost of weed control with the damage expected to agriculture in terms of financial loss (cost-benefit analysis). The simulations for a number of years show that in most years weed control once a year is acceptable.

Like the cost-benefit data, the application of the multi-objective decision method ELECTRE II included the ecological impact of weed control. It identifies the most favourable strategies for different goals (agriculture versus nature conservation - see Table 41). For agriculture, one clearing before winter is an acceptable strategy. The options with one clearing during the growing season and one before winter were also very favourable. When the aim is to minimize adverse ecological impact, the option in which one side and the bottom of the water course is cleared during the growing season was preferred. This alternative is a compromise between maintaining sufficient conveyance capacity and preserving weeds that provide shelter for birds, amphibians and other fauna.

1 Introduction

There are many ditches, brooks, canals and rivers in the Netherlands. All this water plays an important role in daily life and serves many purposes (discharge, water supply, for transport, recreation, etc). The smaller water courses, whether of natural origin or man made, serve to drain the land and to prevent flooding. The larger water courses generally serve as channels conveying water and as shipping routes. The presence of water is essential, but often there are periods with a surplus of water and periods with a deficit. Action is therefore required to control the water and to maintain suitable hydrological conditions. Farming areas require relatively dry conditions that promote workability and heavy crop yields, whereas nature areas require wetter conditions so that the valued aquatic and terrestrial vegetation can be maintained.

In many parts of the Netherlands the water table is shallow. The country's dense network of water courses is primarily to drain the land, but also to supply water to agricultural areas. As a result of these different functions the water levels vary over the year. In winter surface water levels are kept low to encourage quick drainage and in spring they are allowed to rise, to conserve water for dry periods in the coming summer. In Dutch nature areas water levels are often kept high throughout the year in order to preserve wet conditions. All this can have a significant impact on the entire hydrological system of a catchment area. Both the groundwater and the surface water systems play an important role in this respect. A change to one of these systems can have a significant effect on the other. A related aspect is the changing transport capacity of the water courses due to the growth of aquatic plants in the growing season. In turn this phenomenon requires weeds to be controlled in order to maintain the water course's required capacity. Weed control has a great ecological impact. Growing awareness of the link between the environment and water management also underlines the importance of carrying out the weed control in a way that minimizes ecological damage. The feasibility of having various degrees of weed control suitable for agriculture and nature conservation needs to be determined. Certain measures might be less favourable for the hydrological conditions in agricultural areas. An integrated approach is therefore necessary, so that all functions are harmonized.

1.1 Aquatic weed control

In the Netherlands the water boards are responsible for the water management in rural areas. Their main task is to prevent flooding and to supply water for agriculture, and this is often combined with water quality control too. In the last twenty years other interests, such as nature conservation and recreation, have become increasingly important. In the past hundred years the water boards improved water management considerably. At present the water boards maintain approximately 55,000 km of water courses themselves while another 125,000 km are maintained by the land owners under their supervision. Traditionally the task of the water boards has been to maintain the conveyance (transport) function of the water courses. Weed growth in the water courses reduces their capacity to transport water and causes higher water levels to occur. Eventually this could cause unacceptable wet conditions (waterlogging), or even flooding of the land. Maintenance by means of weed control is one of the major activities of the water boards. The yearly costs of weed control in the Netherlands have been estimated as about 200 million Dutch guilders (UNIE VAN WATERSCHAPPEN, 1986).

Excessive weed growth primarily affects the capacity of the water courses to transport water, and results in higher surface water levels. Secondary effects are differences in groundwater levels and evapotranspiration. These hydrological changes could affect the workability of the land and depress crop yields. The environmental impact of weed control involves ecological changes and various secondary effects related to the flora and fauna. Weed control is generally done according to a strict time schedule, slightly modified or improved over the years according to experience. It has usually been done in a way that avoids causing damage to agriculture. In practice, certain problems related to weed control are frequently found (DE JAGER, 1986). They are:

- water courses being choked by aquatic weeds within a very short period;
- seasonal and annual variations in hydrological conditions (irregular wet and dry periods; different discharge characteristics);
- flooding can cause great damage to agriculture areas, so it cannot be risked;
- weed control needs to be carried out as efficient as possible, while trying to minimize the cost of equipment and labour.

A weed control programme is also governed by the equipment available. Heavy weed growth is difficult to clear with light equipment and the result is weed control more frequent than strictly necessary to prevent flooding.

The design of drainage channels used to be based on certain rules, such as the design flow, minimum freeboard and maximum velocity. One single value for the flow rate and flow resistance is commonly used to calculate the required dimension of a water course. This implies that very little weed growth can be tolerated. The design discharge or specific discharge (for instance, 1.2 $l \cdot s^{-1} \cdot ha^{-1}$) often occurs in winter or spring time. In summer the maximum discharge is often less, because more water is retained in the unsaturated and saturated zones. In practice this has led to situations where some weed growth has been tolerated.

Scheduling weed control

To maintain an adequate discharge capacity of the water courses during the summer period, weed control is carried out regularly. Figure 1 shows the important factors and goals that influence the weed control. A channel's discharge characteristics are influenced by the groundwater system (seepage, drainage, storage capacity) and meteorological factors (precipitation and evapotranspiration). The discharge capacity depends on the dimensions of the water course. This means that weed control should already be considered in the design stage of new water courses. The growth of weeds depends on factors such as water quality, water temperature, air temperature and substratum. Other factors such as land use, layout of drainage network and the function of the water course are also important for the required weed control. The goals formulated by the province or water board will assign typical functions to the water courses and try to maintain or improve the ecological potential.

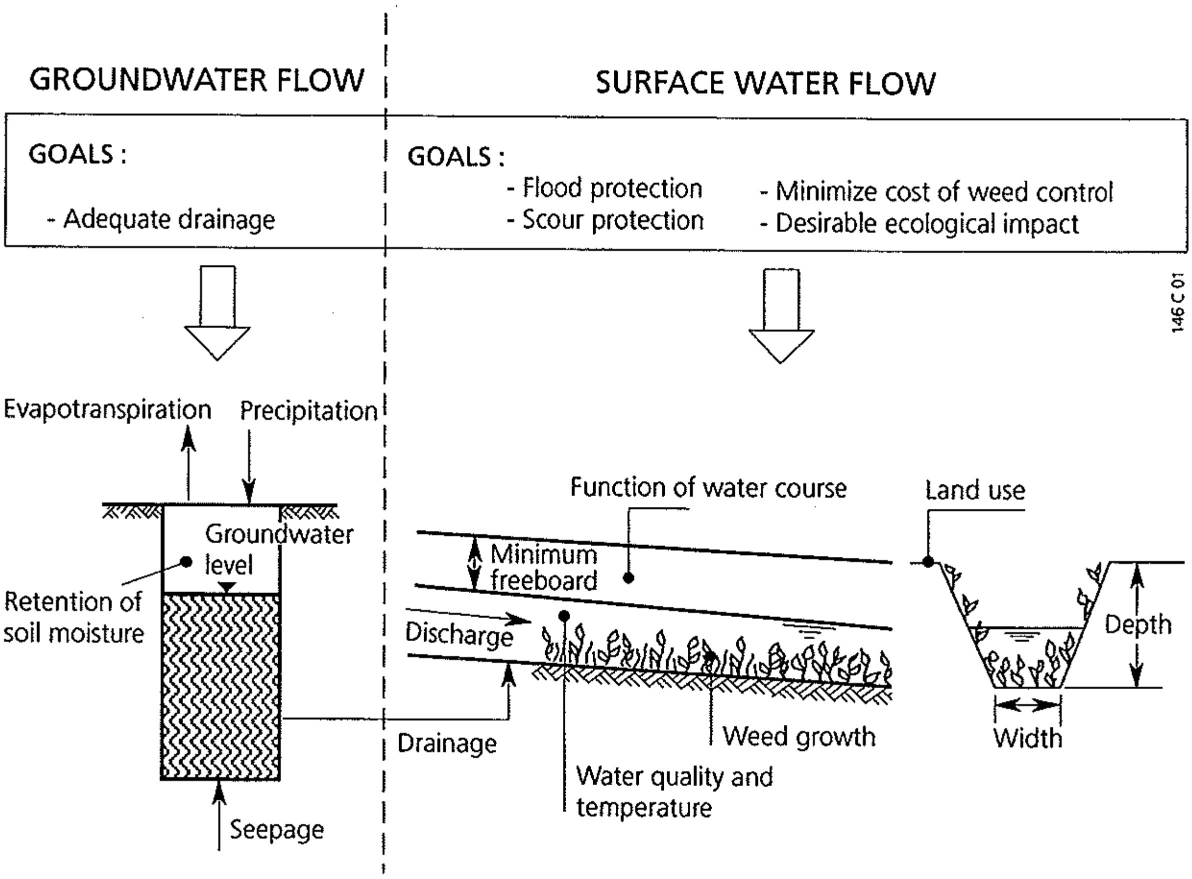


Fig. 1 *The main goals and physical factors related to groundwater and surface water flow that influence the scheduling of weed control.*

1.2 Hydrological modelling for analysing the effects of weed control

Weed control influences surface water levels, flow velocities and groundwater levels. Therefore the amount of drainage and the discharges in a catchment are important. Analytical models are often used to predict the discharges, but the effect of such complex situations, such as weed control, cannot be handled by these models. Therefore a combined surface and groundwater flow model must be used to quantify the variation in discharge, the changes in surface water levels and the changes in groundwater levels. It should be possible to simulate the regional hydrological system as a simplified representation of all the complex processes in reality. Such a simulation model should be physically based, so it can be used to synthesize past hydrological events and to evaluate the effects of changes imposed on the hydrological regime.

A hydrological model to be developed for the analysis of weed control must include the flow of water in the water courses. If the model is to calculate the surface water levels properly, information is needed on the weed growth and the corresponding flow resistance. The groundwater system must be understood, so that groundwater levels and the drainage rates can be calculated. This means that the saturated and unsaturated zones must be considered too. It is also important to keep the model fairly simple, so that hydrologists are able to handle it. Moreover, the computational time should be reasonable.

1.3 Aquatic weed and flow resistance

Open channel flow, being turbulent flow, is characterized by a rapid variation of velocity near the solid boundary and by roughness elements such as aquatic weeds. Therefore the velocity distribution in a cross-section of a channel depends on the roughness elements, type of cross-section and the bed slope. At present no method is available for an analytical solution of the velocity distribution in a channel, not even for the case of a rectangular cross-section. Therefore a variety of semi-empirical formulae exist for computing the flow rate for typical open-channel flow cases. The Manning, the Chézy, and the Darcy-Weisbach formulae are the ones that most commonly appear in today's literature. Despite its shortcomings as pointed out by ROUSE (1965), the Manning formula is the one used most frequently by practising engineers throughout the world. The flow resistance required in any of the three flow formulae cannot be calculated explicitly. It can only be derived from measurements in the field. Hence very many measurements under different circumstances are required to clarify the magnitude of the flow resistance.

Turbulent flow is so complex that a numerical solution is needed to calculate the velocity distribution. The velocity distribution in turn gives the total flow rate and so a roughness coefficient can be derived. A numerical approach using the finite element method has been successfully applied to predict the velocity distribution in sections with concrete lining or a closely mown grass cover (QUERNER, 1981). This model I adopted to include weed obstruction within the cross-section, so it can be used for experiments to estimate the flow resistance.

1.4 Evaluation of weed control strategies

Traditionally, weed control in water courses in the Netherlands has been stipulated by the requirements for farming. This implies increasing the agricultural benefits and minimizing the cost of weed control. The timing of such weed control often conflicts with other interests such as nature conservation. Therefore an integrated approach must be used to establish the schedule and method of weed control that best satisfies all interest groups. Having so many dependable variables complicates the process of determining operational rules. Only limited experiments in the field are feasible, because of the possible risk of flooding.

The combined surface and groundwater flow model I developed was used to carry out numerical experiments with different weed control strategies. It is rather difficult to analyse all these results and to choose the best option. An evaluation technique was necessary to find acceptable options. The method had to consider the environment too. This complicates the decision process even more, because these environmental criteria cannot easily be translated into costs or benefits at the same scale as, for example, in the agricultural sector. This type of problem therefore requires a technique involving multi-objectives. Each objective has its own scale of expressing its appropriateness for a given management option. This decision making, involving a number of options is a frequently used approach which is relatively easy to carry out.

1.5 Outline of research

The research reported here focused on the clearing of aquatic weeds during the growing season. The weed control is considered from two angles, using hydraulic aspects and hydrological models. The first angle is:

> **The estimation of the weed control required for a water course when the expected discharge exceeds the permissible flow rate.**

The drainage from groundwater gives an expected variation in discharge over the growing season. The permissible flow rate is governed by both the growth of weeds and the permissible water levels in the water course. Weed control is necessary during the growing season, when the expected discharge is more than the permissible flow rate.

The second angle is:

The evaluation of different weed control strategies.

A combined surface and groundwater flow model was developed to determine the effect of weed control strategies. The model was run for scenarios assuming different strategies of weed control. The financial loss caused by the extreme hydrological conditions, such as high surface and groundwater levels, was estimated. This loss and the cost of weed control, plus the ecological implications, were evaluated to find acceptable strategies.

The related topics considered within the research objective were:
- the development of a regional surface and groundwater flow model;
- an analysis of such model in order to show the model's ability to simulate crucial processes as fluctuating groundwater levels, discharges and surface water levels;
- the determination of the obstruction caused by weed growth over the growing season;
- the estimation of flow resistance from field measurements and laboratory experiments;
- the use of a numerical approach to estimate the velocity distribution in a cross-section of a water course and hence the flow resistance;
- the variation in specific discharge over the year;
- the application of a multi-objective decision analysis for the evaluation of weed control options using the results from the surface and groundwater flow model and considering the ecological implications.

The surface and groundwater flow model was applied to the Hupselse beek catchment. This experimental catchment (6.5 km^2), for which detailed hydrological data are available, is in the east of the Netherlands (Fig. 35). However it is rather small to evaluate weed control, and its varied relief is uncharacteristic for large areas in the Netherlands. Therefore the case study in which weed control strategies were evaluated, was done in the Poelsbeek and Bolscherbeek catchments. These catchments cover 64 km^2 in total and are some 10 kilometres north of the Hupselse Beek area.

Outline of the remaining chapters

Eight chapters follow this introduction. In Chapter 2 the evaluation approach for weed control strategies is described. The aspects considered in this approach are further elaborated. The open channel flow equations and the necessary roughness coefficients are discussed in Chapter 3. A method is outlined to estimate the velocity distribution in the cross-section of a water course. The flow rate and the flow resistance coefficient can be derived from the velocity distribution. In Chapter 4 the groundwater flow model SIMGRO (saturated and unsaturated flow) is described and the way in which the surface water model SIMWAT and the SIMGRO model were combined into the hydrological model MOGROW. This model is applied to the Hupselse beek catchment (Chapter 5). The comparison between observed and computed variables includes groundwater levels, discharges, actual evapotranspiration and moisture storage in the root zone. The numerical simulations carried out also consider certain special processes such as hysteresis, perched water tables, preferential flow and surface runoff. The application of the MOGROW model to the Poelsbeek and Bolscherbeek areas is described in Chapter 6. The areas are described and the calibration and verification of the model input parameters are discussed. Chapter 7 covers the relevant aspects related to the timing of weed control. Measurements were done to obtain information on the weed growth over time. The flow resistance is derived from field measurements and laboratory experiments. A numerical approach is used to estimate the velocity distribution for cross-sections with weed growth. A procedure is outlined in which hydrological and hydraulic aspects are used to determine the necessary periods of weed control. This procedure is used in an example for three typical hydrological situations. Chapter 8 deals with the case study in which weed control alternatives were evaluated for the Poelsbeek and Bolscherbeek pilot areas. These numerical experiments were evaluated first by comparing the cost of weed control with the financial loss expected mainly to agriculture (cost-benefit analysis). Second the multi-objective decision method ELECTRE II was used, enabling environmental impacts to be included. Both methods show which options are the most promising for the target objectives. Finally, the overall conclusions and recommendations are presented in Chapter 9.

2 Towards an integrated approach for evaluating aquatic weed control

The maintenance of the water courses generally focuses on the control of the weed growth (species and quantity) and also on maintaining the channel dimensions. Weed control is an annual maintenance, while maintenance of the channel dimensions has a frequency of once every three to ten years. The work necessary to maintain the required dimensions of the water courses was not covered in this research.

Aquatic weed control in the Netherlands used to be carried out with the aim of minimizing potential damage to agriculture (SIEFERS, 1985). The removal of aquatic weeds was regarded as a single act, to ensure a sufficient discharge capacity of the water courses during the growing season. This straightforward approach was common practice until recent years and has resulted in water courses that have hardly any vegetation. This frequent clearing of the water courses has affected the aquatic and terrestrial vegetation. It has considerably disrupted nature and the environment. The diverse flora, very common in the past in and alongside the water courses, has declined rapidly (ZONDERWIJK, 1986; VAN STRIEN, 1991). An integrated approach to weed control is therefore essential, to restore a more natural situation. Such an approach must use a decision making framework which explicitly includes these environmental aspects (BOGARDI, 1990).

In Section 2.1 the method adopted for an integrated approach for evaluating weed control is discussed. The hydrological and financial aspects required are further elaborated in Sections 2.2 to 2.4. As part of my research I did a case study in two pilot areas, Poelsbeek and Bolscherbeek, in the east of the Netherlands, to demonstrate the evaluation approach. The hydrological modelling of these areas is described in Chapter 6; the case study is reported in Chapter 8.

2.1 The concept used to evaluate weed control strategies

Weed control primarily influences the water levels in the water courses. The secondary effects are differences in groundwater levels and in evapotranspiration. These hydrological changes affect crop production, soil workability and trafficability. The ecological impact of aquatic weed control is great. The integrated approach proposed in this research for evaluating the impact of weed control is shown in Figure 2. Based on 'professional judgement' or the present weed control

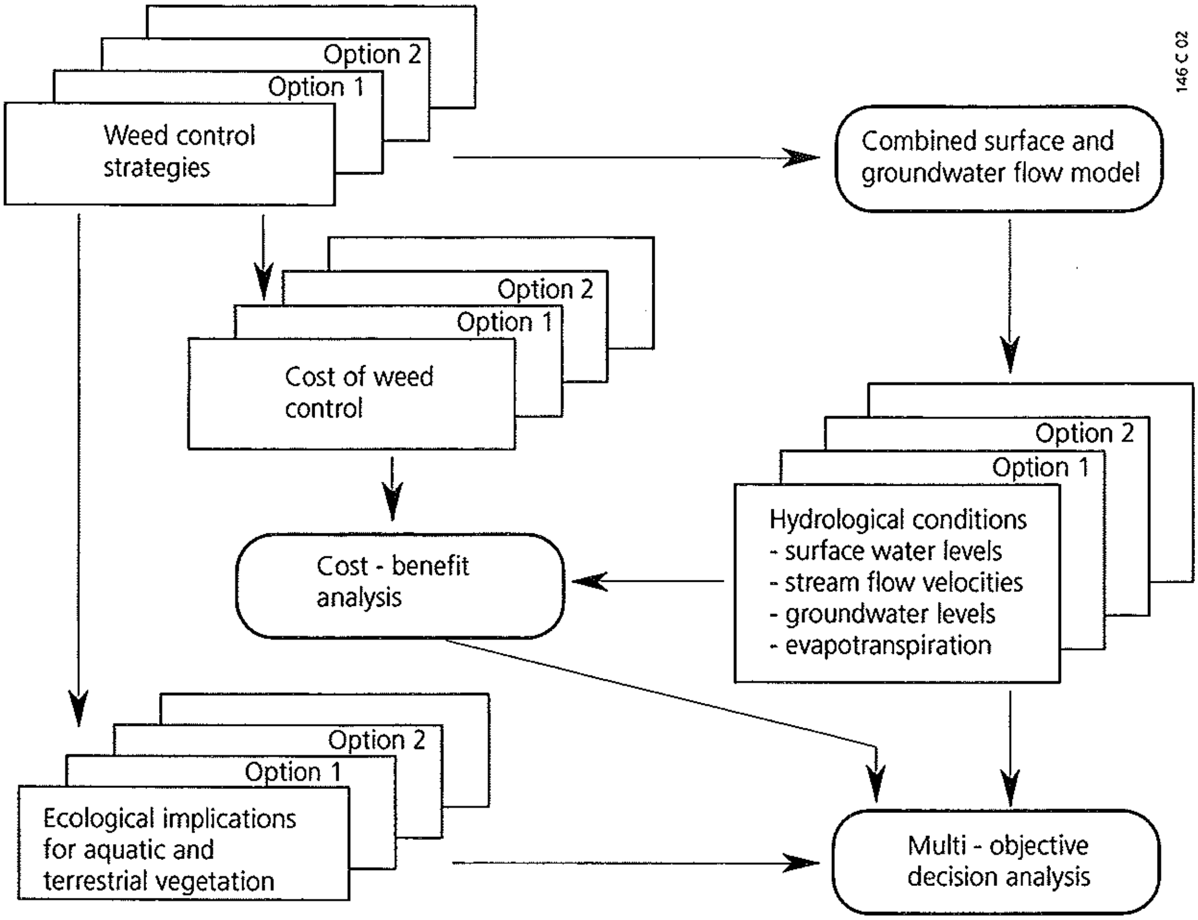


Fig. 2 *Approach to evaluate weed control options, using hydrological conditions in a cost-benefit analysis and incorporating ecological values in a multi-objective decision algorithm.*

programme different weed control strategies (options) are proposed as numerical experiments. A combined surface and groundwater flow model has to be used to quantify the hydrological conditions. This enables the weed control programme to be included and its effects to be ascertained. Simulations with these conditions in the model yield the repercussions on surface water levels, stream flow velocities, groundwater levels and evapotranspiration (Fig. 2). The different options can be evaluated using a cost-benefit analysis and a multi-objective decision analysis.

Cost-benefit analysis

Both the cost of weed control and the financial loss to agriculture can be estimated. This enables the cost of weed control to be compared with the expected financial loss in a cost-benefit analysis. The drawback is that it is very difficult to imply the environmental impacts.

Multi-objective decision analysis

The effects on nature conservation must be incorporated in a multi-objective decision analysis. By considering various weed control options the decision makers (i.e.

the water board), have an opportunity to evaluate these options and to select the most suitable one. The main objective of the evaluation is to quantify the various conditions, so that options can be assessed. The objectives of weed control are to avoid flooding, to prevent groundwater levels becoming too high and to try to minimize the adverse ecological effects. The evaluation remains a quantitative approach in which quality aspects are incorporated only as boundary conditions. The evaluation method as shown in Figure 2 is a first step towards an integrated water management approach to weed control. The major components of the evaluation approach (the decision algorithm, the economic aspects and the modelling of the hydrological system) are discussed in detail in the following sections.

2.2 Modelling the hydrological system

The hydrological parameters important for evaluating weed control strategies are shown in Figure 2. These components require the modelling of surface water flow and regional groundwater flow. The movement and retention of water in such a regional hydrological model should be considered as a simplified representation of the complex processes in reality. When designing this overall modelling approach I used available models for parts of the hydrological system. For the groundwater system I used an unsteady state model that was developed (QUERNER and VAN BAKEL, 1989). For the surface water I developed a model (QUERNER, 1986a), specifically to be combined with the groundwater model. When developing the individual modules and in the stage of integration I kept in mind the need to:

- predict the hydrological processes as accurately as possible. (In general the hydrological processes included in the model and the necessary accuracy of the prediction were defined by the research objective);
- limit the amount of input data needed. (The availability of input data or the time available to measure, collect or estimate input data are often constraints. Because meteorological conditions vary during and over the years the system has to be simulated over a number of years);
- maintain a modular structure so that components of the hydrological system can be analysed independently. (Models available for the saturated zone, unsaturated zone and surface water separately);
- maintain a balance between the detail in which all the processes are simulated in the modules. (When the hydrological model is being used for practical situations in a later stage, it will feed back information to show where further detailed modelling - schematization - is required);

- develop a model for practical applications as well as for research purposes. (A combined groundwater and surface water model is also necessary to solve many of the practical water management problems in the Netherlands).

The important processes included in the model were based on physical hydrological concepts. BECKER and SERBAN (1990) noted the lack of larger-scale physically based models. BEVEN (1989) formulated some fundamental problems in the application of physically-based models to regional practical problems. The equations in such models are based on small-scale homogeneous conditions. Therefore the model schematization requires a transformation from a small-scale to a grid or finite area. The spatial heterogeneity of certain parameters means that it is sometimes doubtful whether the same physical equations can be applied at the scale of the model (finite element or influence area of nodal point). Specifically this applies to parameters or processes that are non-linear in relation to other parameters. Notwithstanding these criticisms, the physically-based approach is the best way to proceed in the field of numerical simulations. The schematization has to include, as much as possible, the scale effects and the proper ways to handle the non-linear processes. The physically-based models are the only ones that can be used in situations with changing conditions which affect the hydrological system. Examples of such changing conditions are land use, groundwater extractions, drainage depth, etc.

2.3 Costs and benefits of weed control

The need for maintenance by means of weed control is evident, but the intensity of weed control is also a matter of costs and benefits. Weed control has always been a substantial item in the annually budget of the water boards. A survey of the water boards revealed that the total annual expenditure on this is about 83 million Dutch guilders (LOORIJ, 1989). Approximately 54% of the total costs are for labour, 34% for the mechanical equipment and the remainder is for administration. Given the 55,000 km of water courses in the Netherlands this comes to an average annual cost of about NLG 1.50 per metre of water course. The length of the water courses to be maintained by the landowners and other organizations is in the order of 190,000 km (UNIE VAN WATERSCHAPPEN, 1986). For water courses in a region with typical sandy soil the costs of one clearing event are given in Table 1.

The actual cost of a single clearing event depends on numerous other aspects such as bank slope, weed species, accessibility, type of machinery, management strategies, etc. The accessibility is governed by factors such as the presence of a maintenance path or the obstructions such as trees and fences.

Bed width (m)	Average ($NLG \cdot m^{-1}$)	Range ($NLG \cdot m^{-1}$)
0 - 1	0.80	0.37 - 3.50
1 - 2	0.67	0.55 - 2.18
2 - 4	0.90	0.54 - 1.92
> 4	0.92	0.57 - 2.14

Table 1 *Costs of weed control, single clearing event, in regions in the Netherlands with sandy soils (UNIE VAN WATERSCHAPPEN, 1986).*

Financial loss to agriculture

The benefits of weed control are the minimization of the financial losses to agriculture. The estimation of hydrological effects on crop production has been considered in great depth in the 'HELP' evaluation method (VAN WIJK et al., 1988). This method has been used to quantify the variation in operational water management in terms of yield depressions (WERKGROEP HELP-TABEL, 1987). These results apply to the changing hydrological conditions over a growing season, which mean I could not use them in my research. Manipulating the clearing dates for weed control may result in different surface water and groundwater levels for short time. For instance, a delay in weed control followed by heavy rainfall may result in high levels of surface water as of groundwater. Such increased levels last for no more than a couple of days or weeks and therefore their effect on the reduction in transpiration of a crop is generally minor. On the other hand, extreme wet conditions can reduce trafficability. If, for instance this results in a delay in the spraying of pesticides, crop production may be considerably depressed. Therefore relating agricultural benefits (or losses) solely to changes in the evapotranspiration or transpiration is inappropriate for the evaluation of weed control measures. Other factors such as farm management activities can have a much greater effect on the financial damage.

Research on the effect of water management on pasture has been reported by PEERBOOM (1990) and POSTMA (1992). They consider (for Dutch conditions) the yield of grass together with aspects such as farm management, cattle grazing, relocating stock, reduced milk yields, etc. They estimated the damage for a number of water management situations, including the occurrence of high groundwater levels. In Chapter 8 their damage function have been combined with other data and used in a cost-benefit analysis for agriculture.

2.4 Multi-objective decision analysis

In the past, decision making was often based on the aim of optimizing a single criterion or objective function, given a set of prescribed constraints. This might mean maximizing the output or minimizing the cost. Today, water resources must be used as efficiently as possible. The economic aspects are important, but the environment must not be overlooked. Since these aspects need action and the consequences may be conflicting, multiple objectives need to be considered in water management problems such as weed control. This means that specific goals need to be formulated in terms of criteria, so that different weed control strategies can be evaluated. Various methods have been developed for multi-objective decision analysis (e.g. GOICOECHEA et al., 1982). I formulated different weed control strategies by varying the frequency of weed control or changing the period in which the work is carried out. This yielded various options to be evaluated. This type of discrete problem can be analysed with the weighted average method ELECTRE (elimination and choice translating algorithm). It is a commonly used evaluation procedure in which a numerical value represents a certain quality for each option. Each criterion is assigned a weight and the procedure is to select the option with the highest value. The method is therefore focused on the ranking of options (ROY, 1977; NACHTNEBEL, 1990). Changing the weights for specific criteria reveals the sensitivity of the given solution. This enables the options to be judged on their merits. The method is discussed further in Chapter 8, where it is used in the case study to evaluate weed control strategies.

3 Physical processes of open channel flow

3.1 Flow formulae

Various conditions for open channel flow have been defined to express the variation in flow rate and depth with respect to time and in space. The flow of water in a surface water system can usually be classified as being gradually varied unsteady flow (CHOW, 1959). This type of flow prevails under the hydrological conditions of a mild flood wave profile and a gradual change in flow depth.

The law of continuity and the dynamic equation for gradually varied flow are discussed in Section 3.1.1. The boundary friction or roughness required in the dynamic equation is evaluated by one of the uniform flow formulae. Some of the equations commonly used for this will be discussed in Section 3.1.2.

3.1.1 Unsteady flow

For a prismatic section the continuity equation is written as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{1}$$

where:
Q = flow rate ($m^3 \cdot s^{-1}$)
A = wetted cross-sectional area (m^2)
x = distance in longitudinal direction (m)
t = time (s)

The dynamic equation for gradually varied flow, often referred to as the Saint-Venant equation is written as:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(\alpha_v \frac{Q^2}{A}) + gA\frac{\partial y}{\partial x} - gAI + gAS = 0 \tag{2}$$

where:
α_v = coefficient to express non-uniform velocity distribution in the cross-section (-)
g = acceleration due to gravity ($m \cdot s^{-2}$)
y = water depth (m)
I = bed slope (-)
S = hydraulic gradient (-)

The first term of Equation (2) is the effect of inertia, the second term is the effect of non-uniform flow, the third term is the effect of hydrostatic pressure, the fourth term is the gravity component and the fifth term is the boundary friction term. The friction term can be written in a form as derived from any friction formulae, such as the Chézy equation or the Manning equation. Equation (2) was derived from considerations of energy, but it could also be derived from expressions for the conservation of momentum in an unsteady flow. The equation is valid for turbulent flow where the vertical velocity component can be neglected. Equations (1) and (2) enable the water movement to be modelled at any location and time within a surface water network with given boundary conditions. They cannot be solved analytically and therefore an approximate solution is necessary in the form of a solution at a number of specified locations and at certain time intervals.

3.1.2 Uniform flow

The flow of water in an open channel encounters a resistance as it flows down a slope. Uniform flow occurs when the water depth, wetted area and velocity remain constant in time and in the direction of motion (Fig. 3). Over the last two centuries a number of flow equations for uniform flow in open channels have been presented. All are semi-empirical, based on field investigations and hydraulic scale models.

The first formula for the resistance to flow in open channels was proposed by Chézy in 1769 as (CHOW, 1959):

$$v_a = C R^{1/2} S^{1/2} \tag{3}$$

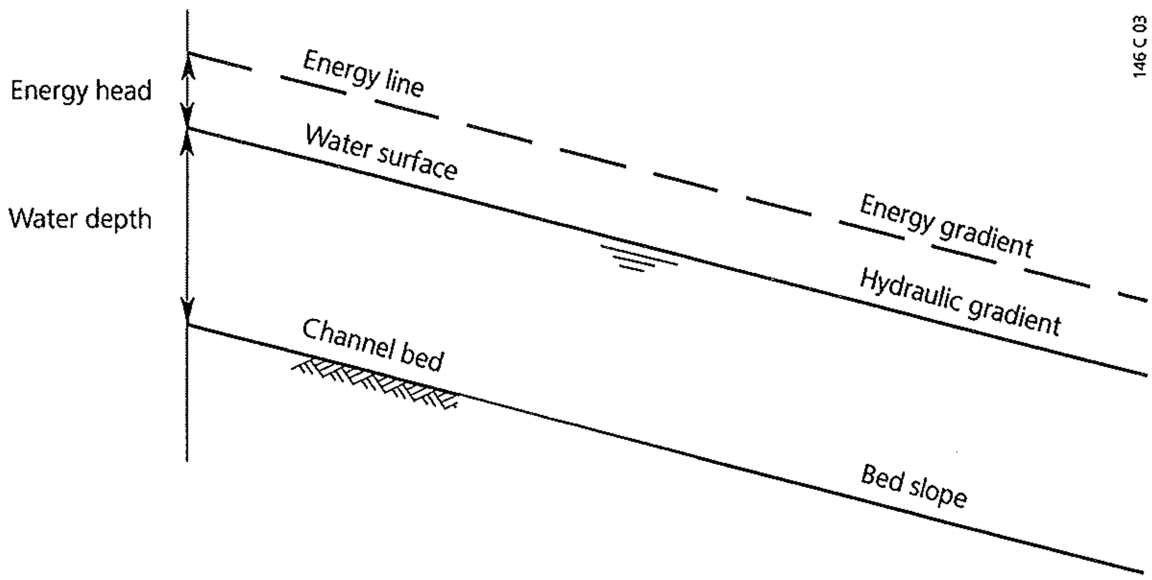


Fig. 3 Uniform flow considers the water depth, wetted area and velocity constant at each section through the channel. This means that energy line, water surface and channel bed run parallel.

where:
v_a = average velocity for wetted area ($m \cdot s^{-1}$)
C = Chézy's coefficient of boundary roughness ($m^{1/2} \cdot s^{-1}$)
R = hydraulic radius, being wetted area A divided by wetted perimeter P (m)
S = hydraulic gradient (-)

Chézy reasoned that the resistance would vary with the wetted perimeter and with the square of the velocity, and that the force to balance this resistance would vary with the area of the cross-section and with the slope. Therefore he assumed that $v_a^2 \cdot R^{-1} \cdot S^{-1}$ would be constant for any one channel, and would be the same for any similar channel. After Chézy's investigation research was done to determine the coefficient C. In principle the conclusion was that C depends on the hydraulic radius as well as on the boundary roughness. For a rough surface the relationship to estimate the coefficient C is given by the White-Colebrook formula:

$$C = 18 \log\left(\frac{12R}{k_e}\right) \tag{4}$$

where:
k_e = boundary roughness height (m)

Another well known flow formula for open channel flow is the Manning equation. It was proposed by various authors between 1880 and 1910 (CHOW, 1959) and written as:

$$v_a = \frac{1}{n} R^{2/3} S^{1/2} \tag{5}$$

where:
n = coefficient of roughness ($s \cdot m^{-1/3}$)

On mainland Europe the factor n^{-1} is often denoted as the k_M value. This value cannot be considered as a flow resistance, but as a conveyance factor or retardance coefficient.

The Darcy-Weisbach formula, developed primarily for flow in pipes, uses the so-called Darcy-Weisbach friction factor (also referred to as Fanning friction factor). The relation between the four commonly used resistance factors C, n, k_M and f is:

$$C = \frac{R^{1/6}}{n} = k_M R^{1/6} = \sqrt{\frac{8g}{f}} \tag{6}$$

where:

g = acceleration due to gravity ($m \cdot s^{-2}$)

f = Darcy-Weisbach friction factor (-)

k_M = boundary roughness coefficient ($m^{1/3} \cdot s^{-1}$)

From the above it can be concluded that the equations merely provide methods for calculating the average velocity, using some resistance and shape factor. Which formula is chosen depends on tradition or convenience. The Chézy coefficient is relatively insensitive to changes in the boundary roughness. A change in the boundary roughness height k_e (Eq. 4) by a factor of 2 results in a change in the Chézy's coefficient C of about 10% (QUERNER, 1985). On the other hand the coefficient of roughness n (Eq. 5) is linearly related to the average velocity.

The hydraulic radius R required in each of the formulae has been the subject of many investigations. This factor has been universally used in the hydraulic calculations relating the shape of open channels and conduits to the average velocity. The use of the hydraulic radius is in fact a poor means for determining the average velocity in a channel, as it is based on the assumption of a uniform distribution of shear along the boundary.

The flow resistance or roughness coefficient required in any of the flow formula should include all aspects which have an influence on the flow rate. In general the flow resistance in a water course is affected by:

- surface roughness and sediments
- shape, density and size of aquatic weeds
- shape of the cross-section and its irregularity
- alignment of water courses (especially bends)
- bed slope
- velocity

The flow resistance cannot be calculated, but is derived from field measurements. The resistance factor for rigid boundaries has been intensively investigated during the last century. However, when the roughness consists of aquatic weeds, the research results are inadequate. The relevant research undertaken over the last 40 years on the flow resistance by aquatic weed growth is discussed in Section 3.2. In Section 3.3 the obstruction by aquatic weed growth is considered in more detail. The flow conditions are discussed and the obstruction is related to the average velocity. In Section 3.4 the procedure to estimate the flow resistance is outlined. I used a numerical model to calculate the velocity distribution and hence the flow rate, for a cross-section with weed growth conditions.

3.2 Review of research on flow resistance of aquatic weeds

In the period 1950 until 1970 important research related to the flow resistance of aquatic weeds was done in the Netherlands and abroad. At first, only field measurements were undertaken to obtain the variation in flow resistance over the growing season. Since 1965 scale models have been used as well, using wooden pegs and plastic strips to simulate the aquatic weeds. This section will describe the research carried out in the Netherlands and abroad which is relevant to the present investigation.

BOS and BIJKERK (1963) found that the flow resistance depends mainly on the variation in water depth. They therefore suggested the relationship:

$$k_M = k_M^r \, y^a \tag{7}$$

where:

k_M^r = so-called reference roughness coefficient ($m^{1/3} \cdot s^{-1}$)

y = water depth (m)

a = constant (-)

The reference roughness coefficient, which differs for summer and winter, is given respectively as $k_M^r = 22.5$ and $k_M^r = 33.8$. They assigned the value 0.33 to a.

FLACH and PIETERS (1966) did many measurements in the field. They estimated the variation of the roughness coefficient k_M over the growing season (Fig. 4). From this figure it can be seen that the major change in roughness coefficient occurs in

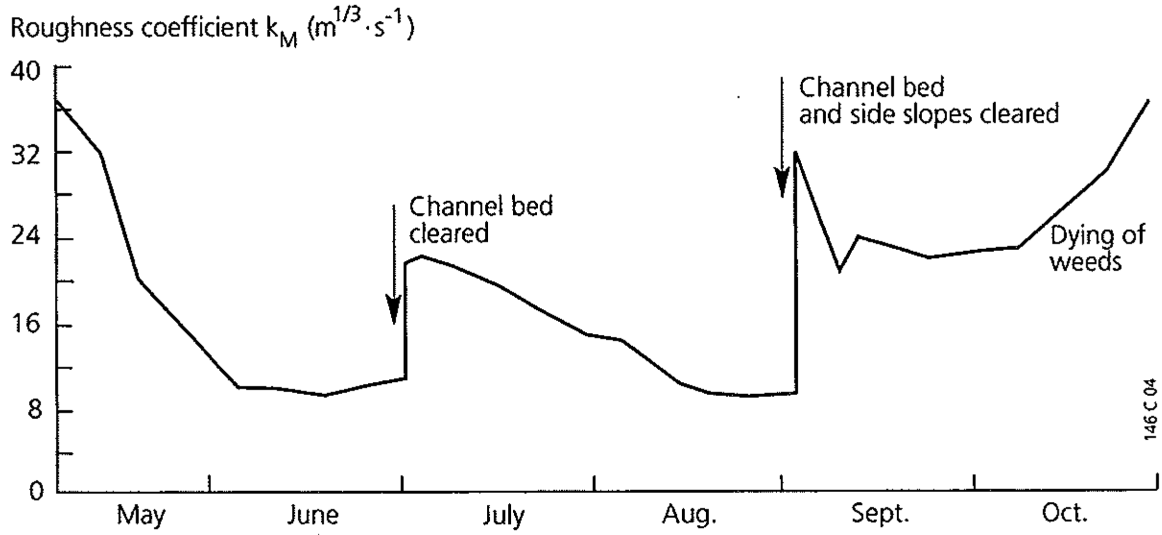


Fig. 4 The variation of the roughness coefficient k_M during the growing season caused by the weed growth and the clearing of weeds (FLACH and PIETERS, 1966).

May and June, giving a variation of k_M from as high as 35 to a value of k_M below 10. BON (1967) introduced an appraisal diagram to determine an appropriate roughness class. The aspects considered in this appraisal diagram are: water depth, aquatic weeds, reduction of the wetted profile, side slopes, obstacles and transport of sediment. In this way a roughness class is derived, which is related to the average flow velocity v_a (Fig. 5). The effect of the velocity on the flow resistance has been reported frequently, but Bon was the first to quantify this important aspect.

In the USA the research related to flow resistance of open channel flow is mainly concentrated on the flow resistance of grass-lined open channels. These open channels often have the function of irrigation canal or serve as storm water outfall from urban areas. They have relatively steep slopes and high velocities often occur for short duration. For design purposes the important parameters are generally the maximum permissible velocity and the flow resistance necessary to cope with a prescribed maximum flow rate. In 1947 the US Department of Agriculture, Stillwater Outdoor Hydraulic Laboratory and the Oklahoma Agricultural Experiment Station presented the results of extensive measurements done in the period 1937 until 1946 (USDA, 1947). From the experiments it appeared that the flow resistance (Manning n) is a function of the product of velocity v and hydraulic

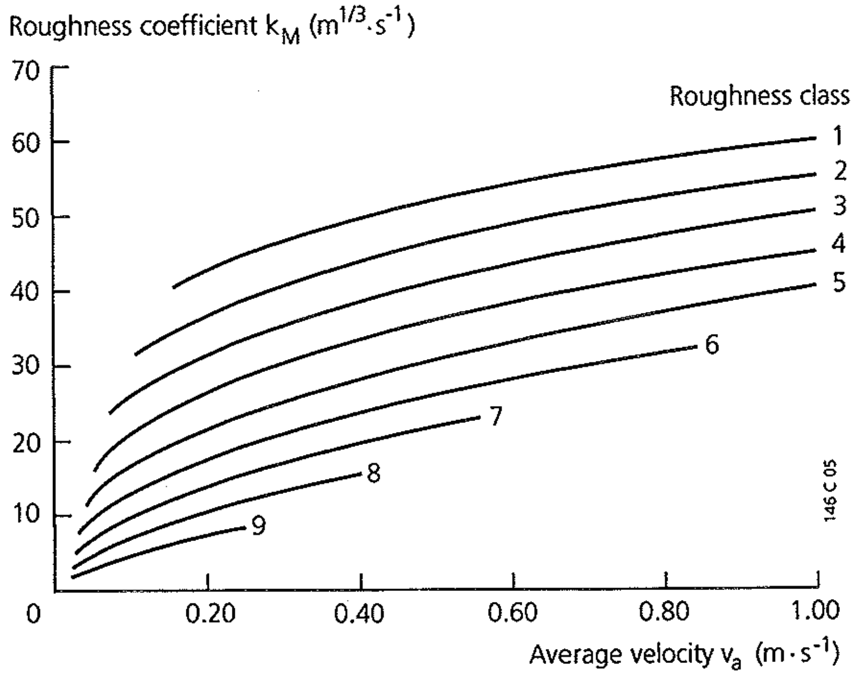


Fig. 5 *The roughness coefficient k_M as a function of the average velocity v_a for the roughness classes 1 to 9. The roughness classes are depicted by factors such as water depth, reduction wetted area, obstacles and transport of material (BON, 1967).*

radius R, often referred to as the n-vR method. The vegetation on the channel bed was classified into five classes ranging between short cut grass and dense/long vegetation. The measurements were carried out in channels having a slope of more than 3%. The theory underlying the n-vR method has never been presented. More recent research (KAO and BARFIELD, 1978) indicates that the n-vR method is only applicable to steep slopes (> 1%). For situations with a very gentle slope - as is the case in the Netherlands - the method is inappropriate, because the hydraulic gradient plays a major role in the magnitude of the flow resistance.

In his research, KOUWEN (1970) quantified the roughness in terms of weed length and the resistance to bending. The principle is that the vegetation is deflected by the velocity (Fig. 6). The effective roughness height (vegetation length) is therefore reduced. The roughness is quantified in terms of velocity, shape, size and resistance of the vegetation to bending. He used the semi-logarithmic resistance relation as (ROUSE, 1965):

$$\frac{1}{\sqrt{f}} = a + b \log \left(\frac{y_n}{k_e}\right) \tag{8}$$

where:

f = Darcy-Weisbach friction factor (-)
y_n = normal depth of flow (m)
k_e = roughness height (m)
a,b = constants (-)

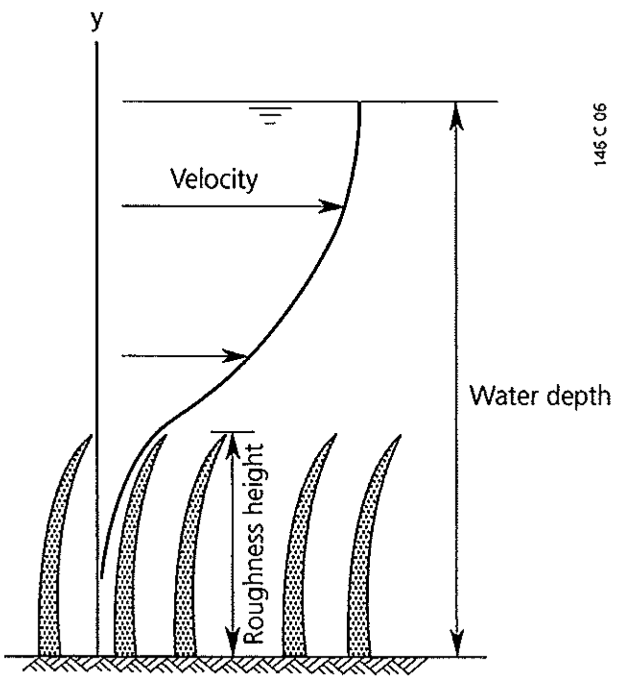


Fig. 6 *Typical measured velocity profile, for situations when the roughness elements are deflected by the velocity (KOUWEN, 1970).*

The vegetation can react to flow in two ways: it will remain erect or prone. Equation (8) can be rewritten for aquatic weeds that are liable to bend, as (KOUWEN et al., 1981):

$$k_M = \frac{\sqrt{8g[a + b \log(y_n/k_e)]}}{y_n^{1/6}} \tag{9}$$

Equation (9) is valid in this form when the hydraulic radius approximately equals the depth of flow. The parameters a and b depend on the shear velocity (Eq. 28) and the situation that either the vegetation remains erect or is prone. Parameter a ranges between 0.15 and 0.29 and parameter b between 1.85 and 3.5 (KOUWEN et al., 1981).

In Germany RADERMACHER (1970) proposed a procedure for designing small water courses in relation to weed growth and maintenance. The method considers the flow resistance as a relative weed obstruction during the course of the year (Fig. 7). A maximum flow rate is defined when no obstruction is present. The reduction of this maximum flow rate is quantified in terms of the growing period of aquatic weeds and the cross-sectional shape. The flow rate for a clear section is given in the form of a nomogram and as a function of the parameters of flow in formulae such as Manning or Chézy. Because of weed growth the flow rate for a section without weed growth, Q_n, is reduced by a 'weed factor' F_w to yield the actual flow rate:

$$Q = F_w Q_n \tag{10}$$

in which:

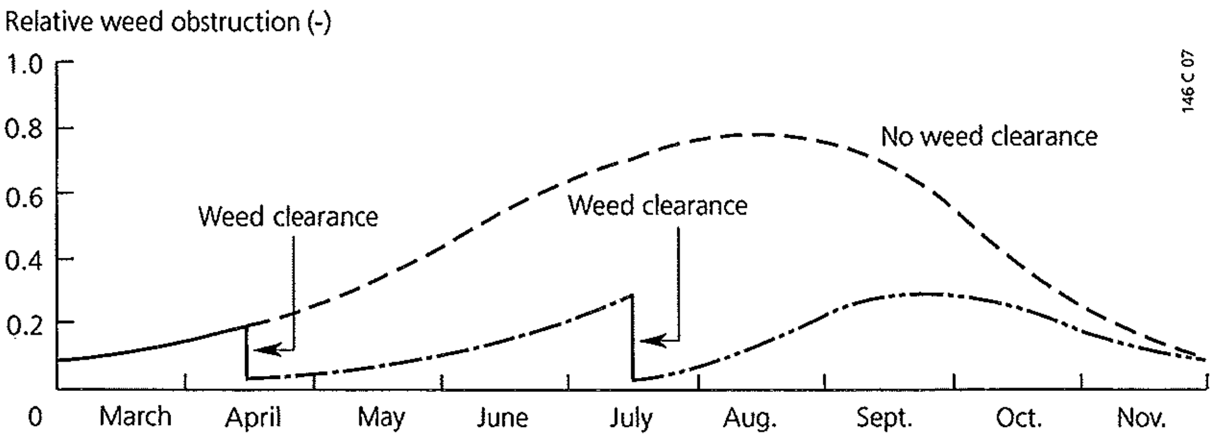


Fig. 7 Change in relative weed obstruction during the summer period (RADERMACHER, 1970).

$$F_w = (1 - \frac{W_r}{F_a})F_z \tag{11}$$

where:

W_r = relative weed obstruction for standard cross-section (-)

F_a = factor to account for different cross-section shapes caused by varying water depth and base width (range 1.0-1.25 for water depth of 0.6–2.0 m)

F_z = factor to account for the variation in side slope (range 0.98-1.02 for side slope of 0.5–2.0)

The procedure uses a nomogram, which is given for a standard cross-section (a bed width of 0.60 m and side slopes of 1:1.5). The factors F_a and F_z express the difference between the actual and the standard cross-section. The relative weed obstruction W_r for the aquatic weeds is a function of (Fig. 8):

$$W_r = f\{T_w, y, z, S\} \tag{12}$$

where:

T_w = period of weed growth (s)

y = water depth (m)

z = angle of side slope (-)

S = hydraulic gradient (-)

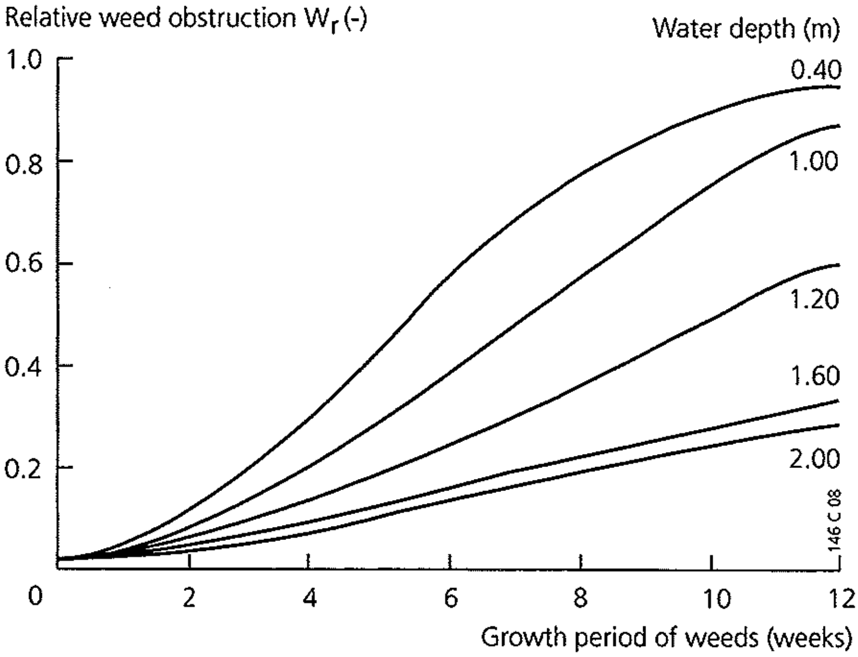


Fig. 8 *Relative weed obstruction dependent on growth period and water depth for a cross-section with a base width of 0.6 m and a bed slope of 0.05% (RADERMACHER, 1970).*

The rate of obstruction as shown in Figure 8 is only given for one period of 12 weeks. The calculation procedure is given in graphical form, which makes the method very simple and easy to understand. The effect of the weeds on the discharge capacity of the channel is calculated as a reduction of the maximum flow rate (factor F_w). For each channel reach one can estimate the permissible reduction from the undisturbed flow rate, before an excessive high water level or damage occurs. On the other hand no explicit form of the flow resistance is given anywhere, which makes it difficult to compare this method with other results.

Different authors have presented the relation between the relative weed obstruction and the roughness coefficient (Fig. 9). Measurements performed in 12 ditches by NITSCHKE (1983) gave roughness coefficients shown in Figure 9. PIETERS and FLACH (1966) obtained roughness coefficients from various measurements, the limits (minimum and maximum range) are also shown in Figure 9. These results are very close to the results reported by Nitschke. A similar approach was used by PITLO (1990). He obtained a linear relation between the relative weed obstruction and the roughness coefficient (Fig. 9). These measurement were obtained in three water courses, ranging in water depth of 0.5–14 m and average velocities in the range of 0.05–0.25 m·s^{-1}. All the results as shown in Figure 9 give quite a range of roughness coefficients. Taking 40% obstruction would give a k_M between 13 and 22. It would mean a difference of about 40% in the capacity of the water course. This large variation is unacceptable.

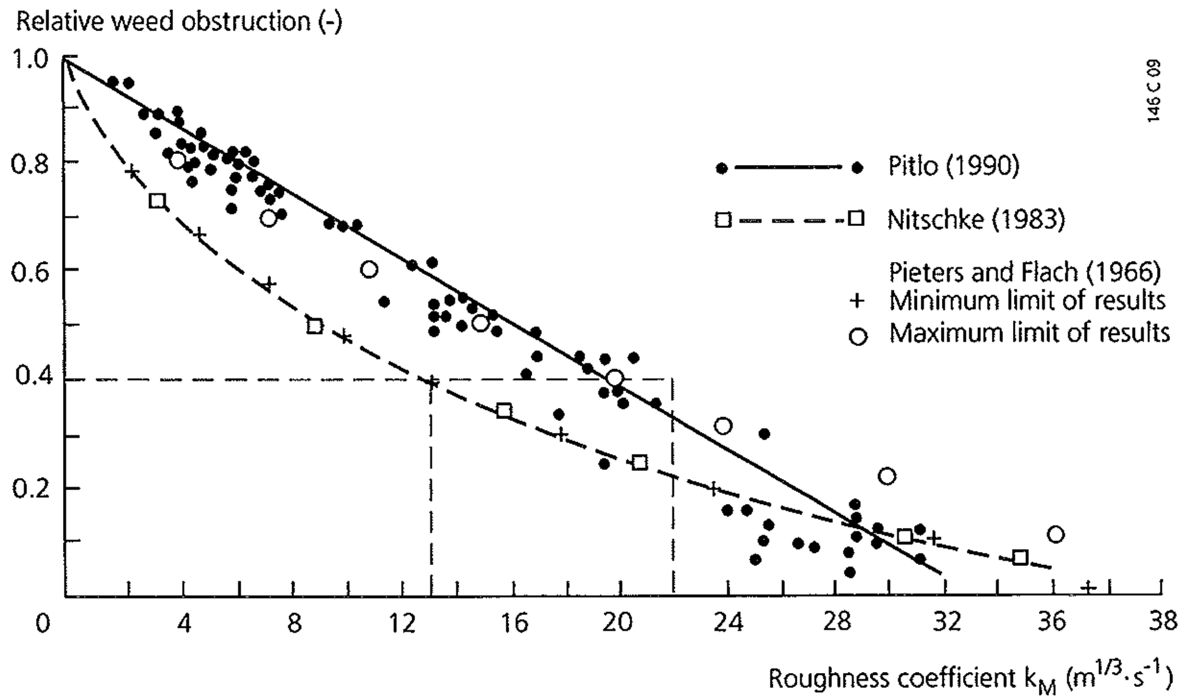


Fig. 9 *Relation between the roughness coefficient k_M and the relative weed obstruction (area weeds divided by wetted area) obtained by different authors.*

Conclusion
In the literature the roughness is very often given as a single value. In reality, the value is very variable and depends on factors such as velocity and rate of obstruction. Hence there must be a relation with the growing period of aquatic weeds. At present it is impossible to make a proper estimate of the flow resistance itself for these factors and different types of aquatic weeds. Therefore field measurements under different flow conditions are still necessary.

To date, flow resistance has always been related to the unobstructed (clear) cross-section. For water courses with dense weed growth this manner of relating and using the roughness coefficient is not ideal. The differences in roughness coefficients obtained are too large (Fig. 9). The rate at which the water course is overgrowing with weeds is very important for establishing the required frequency of weed control. This aspect is not considered in any of the procedures discussed. A suitable method should focus more on the rate of obstruction, together with appropriate roughness coefficients. In Chapter 7 such a method will be outlined, with field and laboratory experiments to support it.

3.3 The theory relating to the obstruction to flow caused by aquatic weeds

Vegetation in an open channel retards the flow of water by causing a loss of energy through turbulence and by exerting additional drag forces on the moving fluid. Because of the complex nature of the flow system and the variety of conditions present, it is nearly impossible to develop an analytical flow model based entirely on theoretical considerations. Therefore, much of the work done in the past has been empirical or semi-empirical. The flow resistance for a certain weed growth is a function of many variables including flow velocity, vertical distribution of the weeds, roughness of the solid boundary and structural properties associated with the stems and leaves of the plants. Two distinct situations were considered to analyse the flow resistance, as shown in Figure 10. I will first discuss the case of submerged aquatic weeds (Section 3.3.1) and then the case of pronounced weed obstruction on the side slopes of the cross-section (Section 3.3.2).

3.3.1 Water flowing over submerged aquatic weeds

It has been found experimentally that water flowing over aquatic weeds can be characterized in principle by three zones (GOURLAY, 1970). The first zone extends upwards from the bed, within the weeds (Fig. 10a). Here the flow velocity is very

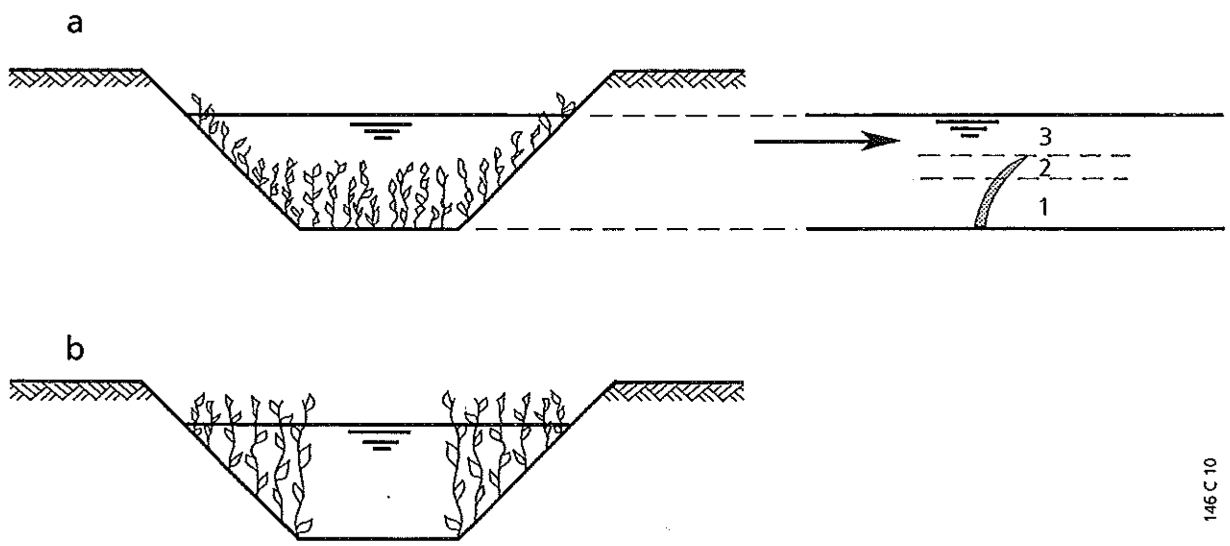


Fig. 10 *Typical weed growth conditions relevant for the flow resistance.*
a) Water flowing over submerged aquatic weeds, which deflects the plants (for description of the three zones see text)
b) Dense weed growth on side slopes, which protrudes above the water surface

small, with little to no vertical variation. The vegetation is not deflected and remains stationary. The flow velocity in this region is dependent on the amount of weeds (obstruction) and on the shear at the solid boundary. The second zone is characterized by the trailing effect of the weed caused by the flow velocity; a waving motion can be seen. The top of this region normally coincides with the upper surface of the bent-over weeds. In this zone there is a marked vertical variation in velocity. In the third zone there are no weeds and a logarithmic variation in velocity occurs. This variation can also be observed in open channels with a rigid boundary roughness (ROUSE, 1965).

Depending on the magnitude of the velocity three phases can be distinguished related to the characteristics of the aquatic weeds. With increasing velocity the following situations can be observed: the aquatic weed remains vertical, the weed is bent over by the flow velocity, and the weed is pushed flat against the bed. These three phases coincide with the manner the sediment moves in rivers: no sediment transport, the sediment moves over the bed and forms a rippled surface, and the sediment is transported nearly all in suspension. The essential aspect that aquatic weed and sediment have in common is that the roughness elements can be changed by the velocity. The flow resistance is therefore a function of the flow velocity and cannot be treated as a constant. Any solution procedure for this type of condition should yield a relationship between the velocity field and the characteristics of the aquatic weed. For this condition the research done by KOUWEN (1970), as discussed in Section 3.2, is applicable. It considers the effective height of the weed in some form (roughness elements). The height of the roughness elements can be estimated from information on the velocity and characteristics of the weed.

3.3.2 Weed obstruction and average velocity

Taking the situation with dense weed growth as shown in Figure 10b, then the balancing of the forces for a control section can be considered. Water flowing in an open channel exerts a longitudinal shear force on the wetted perimeter of the cross-section. Consider a channel reach as a control volume between sections 1 and 2 (Fig. 11). Because of momentum, the sum of the forces in the x direction is equal to zero, or:

$$\sum F_x = 0 \qquad (13)$$

This equilibrium equation can be derived, by balancing the retardance shear force at the boundary against a propulsive force acting in the direction of flow (PETRYK and BOSMAJIAN, 1975). The propulsive force is due to the weight of the flowing water resolved down a slope. The pressure forces in the x direction cancel each other. The remaining forces are gravity, shear forces on the boundary caused by viscosity and boundary roughness, and drag forces on the plants. Inserting these forces in Equation (13) gives:

$$\gamma ALS - \sum_{i=1}^{N} D_i - \tau_a PL = 0 \qquad (14)$$

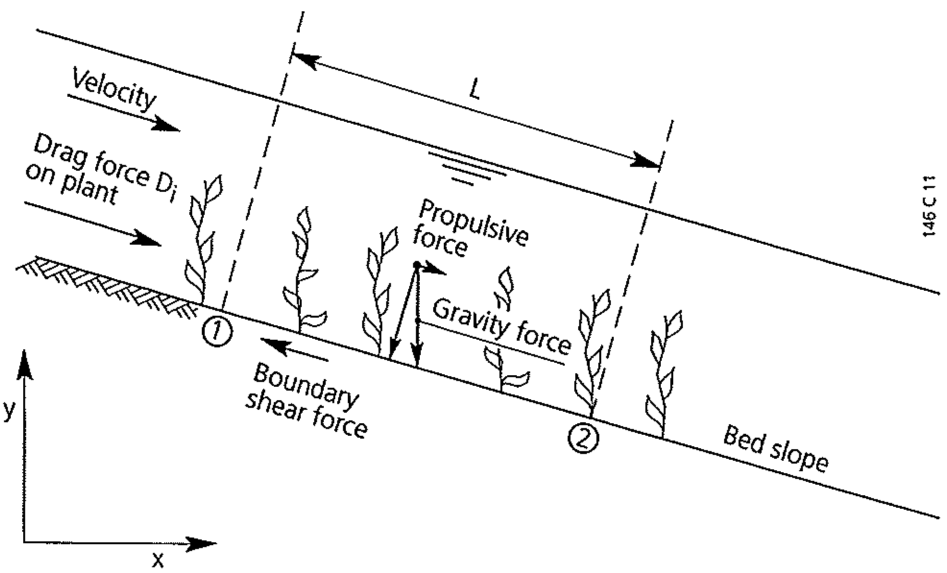


Fig. 11 Typical section of a water course with weed growth and the relevant forces acting within this section.

where:
γ = specific weight of liquid (N·m^{-3})
A = wetted area (m^2)
L = length of channel reach (m)
S = hydraulic gradient (-)
D_i = drag force on a plant (N)
N = number of plants (-)
τ_a = shear stress per unit area on the channel boundary (N·m^{-2})
P = wetted perimeter of channel (m)

The boundary shear is generally not uniformly distributed over the wetted perimeter. Two factors tend to make this force non-uniform: the cross-sectional shape and the existence of a free surface where the shear stress is negligibly small. The drag force on each plant may be described by:

$$D_i = \frac{C_d \, \gamma \, v_i^2 A_p}{2g} \tag{15}$$

where:
C_d = drag coefficient for the vegetation (-)
v_i = average approach velocity to the ith plant (m·s^{-1})
A_p = projected area of the ith plant in direction of current (m^2)

The drag coefficient is dependent on the regime of flow around the obstruction. In general the drag coefficient is about 1.2 (single cylinder). In an arrangement with multiple obstructions, interference occurs as reported by LI and SHEN (1973), and the drag coefficient is then in the order of 1.1.

Equation (14) was proposed by PETRYK and BOSMAJIAN (1975) for situations in which the vegetation is partially submerged and more or less uniformly distributed over the entire cross-section. When aquatic weeds are present only in parts of the cross-section (Fig. 10b), then on the edge of the weeds and the open water, the variation in velocity causes turbulent eddies which should be accounted for in Equation (14). This dissipating action can be so important that the effective boundary shear stress becomes very small (TEMPLE, 1980). These turbulent shear stresses are discussed further in Section 3.4.3. The same principle as formulated in Equation (14) was used by VAN IEPEREN and HERFST (1986). They calculated the drag coefficient C_d on the weed obstructions in the water course to be in the order of 5-25, instead of 1.1 to 1.2. They could not explain the high values, but I assume this is caused by turbulent shear stresses present on the interface of the obstructed and unobstructed part, which were ignored.

For the densely vegetated parts of the cross-section only, one can use Equation (14). The shear stress on the boundary is negligible (3rd term of Eq. 14) and so Equation (14) can be written as:

$$gAS = \frac{\sum C_d v_i^2 A_p}{2} \tag{16}$$

In Equation (16) all the approach velocities have been quantified. Only when the velocity distribution is known, can this equation be solved. The approach velocity is raised to a square, which means that the approach velocities cannot be replaced in this form by the average velocity. This important aspect has been ignored by numerous authors. The way to include the effect of the velocity variation is by using a coefficient, in a similar way as is common to express the velocity head of an open channel, being $\alpha_v v^2/2g$ (CHOW, 1959). Using the energy coefficient to account for the variation in velocity, one can write Equation (16) as:

$$v_i = \sqrt{\frac{gAS}{0.5\sum \alpha_v C_d A_p}} \tag{17}$$

Now one can estimate the velocity in terms of vegetation density, but only for those parts of the channel where weed growth is present (Fig. 10b). In Section 3.4 this equation is used to give an initial estimate of the velocity in the procedure to calculate the velocity distribution by means of the finite element method.

3.4 Estimating flow rate and flow resistance using a numerical approach

From the literature review and the theoretical consideration of flow resistance (Sections 3.2 and 3.3) it is clear that there is still no proper way to estimate the flow resistance from certain state variables. At present the flow resistance can only be derived from field measurements. Having measured the flow rate and the slope of the water surface, then one can estimate the flow resistance from the cross-sectional information, using a flow formula (e.g. the Chézy or Manning formulae: Eqs 3 or 5). Thus many measurements obtained under different circumstances are required to get an insight into the flow resistance.

3.4.1 Procedure

Turbulent flow is a complex process which cannot be precisely described. Yet vegetation comes in various shapes and sizes. The process becomes even more complicated because the local velocity near the weeds affects the obstruction by influencing the bending of the weeds. Given the complexity of turbulent flow, a numerical solution will be the only practical method to calculate the velocity distribution and hence the flow resistance, provided that the remaining unknown parameters can be modelled by means of empirical relations. GERARD (1974) used the finite element method to analyse turbulent flow and secondary currents in non-circular conduits. I developed the model SIMVEL (SIMulation of VELocity distribution) using part of the method Gerard outlined, to calculate the velocity distribution in open channels with a rigid boundary roughness (QUERNER, 1978). Even for compound sections differing in roughness a good representation of the measured velocity distribution was found (QUERNER, 1981). In the case of open channels with aquatic weeds this method must be adjusted to allow for weed obstructions within the cross-section and not only along the solid boundary.

The SIMVEL model enables the velocity distribution to be estimated. The velocity and thus the flow rate can be calculated for a specific weed growth and water surface slope. From these results the flow resistance follows, using one of the open channel flow formulae. The procedure is shown schematically in Figure 12. It is not intended to replace field measurements entirely. The model should first be modified to take account of aquatic weeds and be verified with measured data. If the model gives satisfactory results under different circumstances, then it can be used for specific situations for which no field measurements are available.

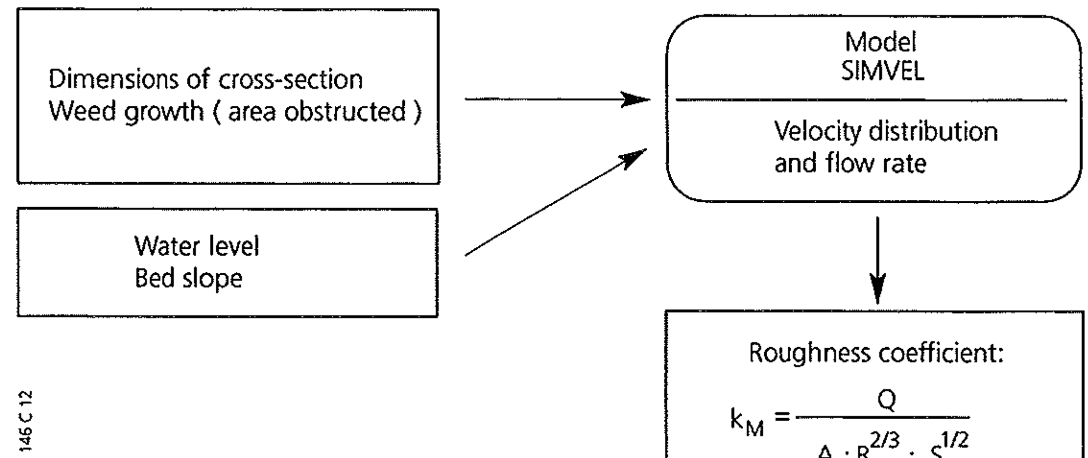


Fig. 12 *Schematic presentation of procedure to calculate flow resistance by means of a numerical method which estimates the velocity distribution in a cross-section. Nomenclature: k_M=roughness coefficient; Q=flow rate; A=wetted area; R=hydraulic radius and S=bed slope.*

The numerical model can also be substituted for some of the measurements of velocities in the field. The flow rate in a water course can be obtained by measuring the flow velocity in a cross-section. The model then calculates the velocity for those parts of the cross-section for which no measurements have been carried out. The resulting velocity distribution over the entire cross-section is used to estimate the flow rate.

Secondary currents

The procedure for calculating the velocity distribution in a cross-section of a water course will be described in Section 3.4.2. The verification of the model and the calculation experiments are discussed in Section 7.2.2. The model takes only the primary velocity component into account, not the secondary currents in the plane of the cross-section. The most pronounced effect of secondary currents is the location of the maximum velocity. Contrary to expectations, the maximum velocity does not occur at the water surface in the middle of the channel, but somewhat lower. This displacement of maximum velocity to below the water surface is mainly due to the drag exerted by the sides of the channel. Frictional losses at the sides reduce the energy and thus the head of the water here, with the result that the energy level at the sides is slightly lower than near the centre of the channel. The energy level must be the same over the entire cross-section, so the result is that the water level at the sides will become slightly higher than in the middle (Fig. 13). This lateral elevation of the water level causes secondary currents to be generated, which travel inward along the water surface from either side to the center. This water then descends near the centre, flows outward along the bed to either bank and upwards along the sides (Fig. 13). The inward surface drift consists of water from a region of minimum velocity and therefore has the effect of reducing the surface velocity. In the analysis these secondary flows were neglected, as they are small compared with

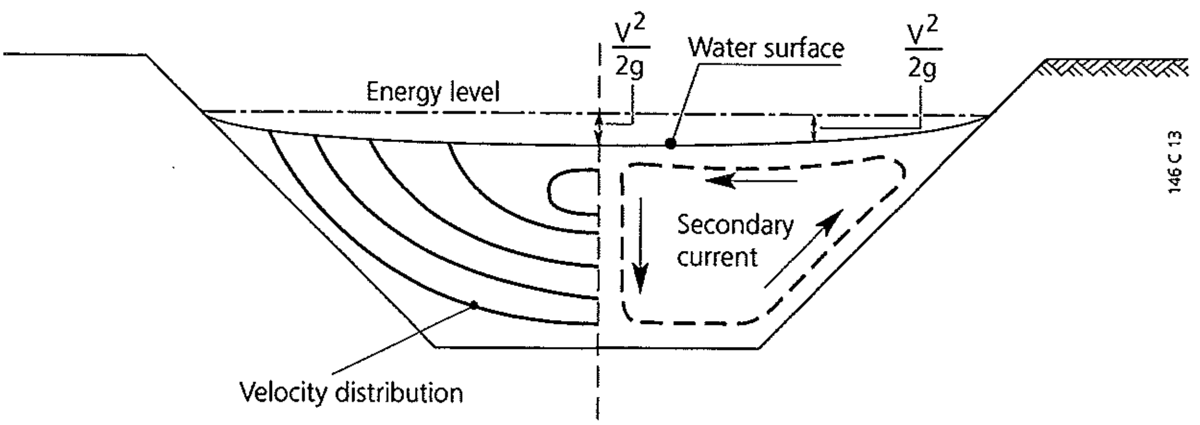


Fig. 13 *Channel cross-section showing typical velocity distribution and secondary currents. The velocity head, $v^2/2g$, in the middle is greater than at the sides, the result is a curved water level and this in turn gives secondary currents.*

the primary flow. The secondary flow is in general 0-5% of the average primary velocity, depending on shape, roughness and size of the cross-section (QUERNER, 1981). This means, that a satisfactory practical analysis of the velocity distribution in open channels can be obtained when secondary flows are neglected.

3.4.2 The finite element approach for calculating velocity distribution

The finite element method, which is formulated in terms of a variational approach, is an extension of the classic Ritz or Galerkin method. Complex configurations can be assembled from relatively simple element shapes. This enables non-uniform boundary conditions, such as different roughnesses, to be specified. The velocity distributions can then be modelled for an open channel with irregular boundary conditions.

Equation of motion for turbulent flow

The method of variational techniques uses certain broad minimum principles and can be formulated as follows: the motion of an incompressible fluid satisfying the continuity equation is such that its function is minimized. This principle was first stated by Helmholtz (LAMB, 1932). He provided the governing function for turbulent steady flow, which can be written as:

$$\phi_m = \int_A \varepsilon[(\frac{\partial v}{\partial x})^2 + (\frac{\partial v}{\partial y})^2]\mathrm{d}A - 2\int_A \rho g S v \,\mathrm{d}A \qquad (18)$$

where:

ϕ_m = excess of dissipation
ε = eddy viscosity ($N{\cdot}s{\cdot}m^{-2}$) (see Section 3.4.3)
v = velocity ($m{\cdot}s^{-1}$)
ρ = fluid density ($N{\cdot}s^2{\cdot}m^{-4}$)
g = acceleration due to gravity ($m{\cdot}s^{-2}$)
S = hydraulic gradient (-)

The first term in Equation (18) can be interpreted as the total kinetic energy of the moving fluid, whereas the second term is related to the amount of work done by an impulsive pressure starting the motion from rest.

Finite element solution

The basic concept of the finite element method is that the medium can be considered as an assemblage of individual elements. The elements are usually chosen to be triangular in shape, so they can be arranged in a variety of ways to represent exceedingly complex shapes. Initially each element is considered independently, and the equations for each element are assembled. Provided that continuity of behaviour between adjacent elements can be assured, the solution of the equations should converge upon the correct numerical solution as the finite element mesh is refined. General background information about the finite element method applied to fluid flow can be found in CONNOR and BREBBIA (1976) or HUEBNER (1975).

A quadratic displacement function for the velocity is used to describe the steep velocity gradient near the solid boundary. This variation of velocity within the element can be represented by the polynomial:

$$v = \alpha_1 + \alpha_2 x + \alpha_3 y + \alpha_4 x^2 + \alpha_5 xy + \alpha_6 y^2 \tag{19}$$

with the α_i parameters known as the generalized coordinates, which will specify or fix the magnitude of the prescribed distribution of the velocity. With this quadratic displacement function and one degree of freedom per node, a 6-node triangular element is required, as shown in Figure 14.

Equation (19) can be written in matrix notation and evaluated for each of the six nodes, giving the expressions:

$$v = \{A\}\{\alpha_i\} \qquad (i = 1 - 6) \tag{20}$$

and

$$\{v_i\} = [B]\{\alpha_i\} \tag{21}$$

where:
$\{A\}$ = vector of polynomial terms
$[B]$ = matrix of polynomial terms, evaluated for each node of an element

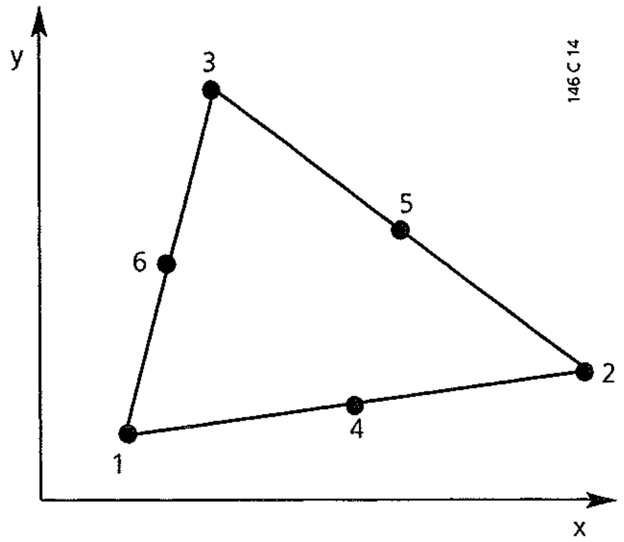


Fig. 14 *Quadratic two-dimensional model for triangular element with six nodes to estimate the variation of velocity within an element (numbering corner and mid-side nodes anti-clockwise).*

Substituting Equations (20) and (21) in Equation (18) and rearranging yields:

$$\phi_m = \{v_i\}^T [K_e] \{v_i\} - 2 \{F_e\}^T \{v_i\} \tag{22}$$

where:
$[K_e]$ = fluid element rigidity
$\{F_e\}$ = external forces on element

The fluid element matrix expresses the relation between the nodal velocities and the associated forces. It is a function of the geometry and eddy viscosity within the element. Instead of a constant value for the eddy viscosity per element, a linear variation is taken because it also increases rapidly near a solid boundary. The vector $\{F_e\}$ contains the external forces acting on the element.

For each element the function defined by Equation (22) must be minimized with respect to each of the undetermined parameters. They are represented by the values of the velocity at the nodes for a typical element. To obtain the general equations it is only necessary to add the contributions of all elements. The final system of simultaneous equations can be written as:

$$\frac{\partial \phi}{\partial v} = [K_t] \{v\} - \{F_t\} = 0 \tag{23}$$

where:
$[K_t]$ = assembled fluid element rigidity matrix
$\{F_t\}$ = assembled vector of all external forces

The above expression establishes an equilibrium condition at each node. The matrix $[K_t]$ is the fluid rigidity of an element connecting the nodal forces with the nodal velocities $\{v\}$. The vector $\{F_t\}$ are the external forces acting on the element.

External force

Equation (22) requires the external force $\{F_e\}$, caused by the bed slope (propulsive force in Fig. 11), which is equal to the hydraulic gradient in the case of steady flow. When aquatic weeds are present the propulsive force on an element is reduced by the obstruction force of the weeds. Using the obstruction force D_i given by Equation (15), one can write the external force $\{F_e\}$ as:

$$\{F_e\} = [L_e] (\rho g S A_e - D_i) \tag{24}$$

where:
$[L_e]$ = interpolation matrix (-)
S = hydraulic gradient (-)

D_i = obstruction by aquatic weeds for element A_e per unit length (Eq. 15) (N)
A_e = area of finite element (m^2)

Boundary condition

Once the final matrix is assembled the boundary conditions must be applied so that the simultaneous equations can be solved. The water surface and lines of symmetry are not constrained in any way and no special boundary conditions need to be specified along them. When no weeds are present near the solid boundary, there is a rapid variation of the velocity, requiring a large number of elements to model this sudden increase. In this region of sudden increase in velocity the flow can be considered as two-dimensional. The logarithmic velocity distribution formula (Equation of Prandtl and von Kármán) can be used for this region (Fig. 15):

$$v_b = (5.75 \log_{10}(\frac{y_b}{k_s}) + 8.5)v_*^1 \qquad (25)$$

where:
v_b = velocity at a distance y_b from the boundary ($m \cdot s^{-1}$)
y_b = distance from the boundary (m)
k_s = Nikuradse's sand grain roughness height (m)
v_*^1 = local boundary shear velocity ($m \cdot s^{-1}$)

The boundary shear velocity required in Equation (25) is defined as:

$$v_* = \sqrt{\frac{\tau_n}{\rho}} \qquad (26)$$

where:
τ_n = shear stress at boundary node n ($N \cdot m^2$)
ρ = fluid density ($N \cdot s^2 \cdot m^{-4}$)

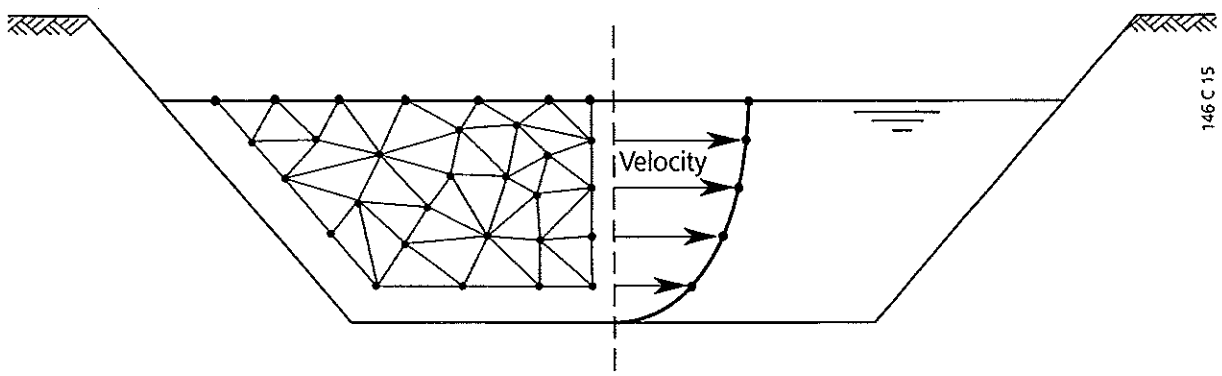


Fig. 15 Typical finite element network for a cross-section of a water course to calculate the velocity in each nodal point. The distance between the boundary and the first internal node uses the logarithmic velocity distribution equation (Eq. 25).

The boundary shear velocity is not a real velocity, but is related to the real fluid velocity, which would give rise to a shear stress τ_n. The calculation of the shear stress at each boundary node point, τ_n, has been approximated according to:

$$\tau_n = \frac{f}{8\rho v_l^2} \tag{27}$$

where:
v_l = local averaged velocity ($m \cdot s^{-1}$)
f = Darcy-Weisbach friction factor (-)

The local average velocity is the average velocity in the vicinity of node n. Inserting Equation (27) into (26) gives an expression for the boundary shear velocity as:

$$v_* = v_l \sqrt{\frac{f}{8}} \tag{28}$$

The friction factor f in relation to Manning's roughness or the Chézy coefficient is given by Equation (6).

3.4.3 Turbulent flow

Depending on the viscosity effect relative to the inertia effect, the flow may be laminar, turbulent, or transitional. In turbulent flow, the water particles move in irregular paths which are neither smooth nor fixed but which in the aggregate still represent the forward motion of the entire stream. There are also many secondary motions, in addition to the broad motion of the fluid, which can be considered random in nature and which affect the mixing of adjacent portions of the fluid. The approach used to analyze turbulent flow is semi-empirical based on mean quantities. A number of relationships applicable to certain types of turbulent flow have been derived. These relationships are largely based on von Kármán's similarity theory, the Prandtl mixing length, and the Taylor vorticity-transport hypotheses (HINZE, 1959). Unfortunately, from these theories it is impossible to derive relationships that accurately apply to the behaviour of the fluid throughout the flow. Prandtl (HINZE, 1959) related his momentum transfer hypothesis to the physical model of a fluid mass moving through a distance L_m normal to the flow streamlines. He called this distance the mixing length. The eddy viscosity depends on the velocity and also on the distance from the boundary and therefore cannot be regarded as a physical constant characteristic of the fluid. The eddy viscosity is a measure of the turbulent shear stress between two layers of fluid moving parallel to each other and the product of the density and velocity gradient. The shear stress results from the exchange of momentum between the layers because of transverse turbulent fluctuations in velocity. The turbulent shear stress can be written as:

$$\tau_t = \varepsilon(\frac{\partial v}{\partial y}) \tag{29}$$

where:
ε = eddy viscosity (N·s·m^{-2})

The eddy viscosity can be written as:

$$\varepsilon = \rho L_m^2 (\frac{\partial v}{\partial y}) \tag{30}$$

where:
L_m = mixing length (m)

It is common practice to assume that the eddy viscosity is dependent upon its position in the channel, using a semi-empirical approach. The simplest assumption is constant, which would be a reasonable first approximation for free turbulence. As we approach a rigid solid boundary the eddy viscosity tends to zero. In the case of a plane-parallel flow close to a boundary, the hypothesis that the eddy viscosity is proportional to the distance from the boundary leads to good results. SCHLINGER (1953) proposed this type of equation for the eddy viscosity as:

$$\varepsilon = K v_* \frac{(y - y_b) y_b}{y} \tag{31}$$

where:
K = von Kármán's constant
y = total water depth of the channel (m)
y_b = distance from the boundary (m)

Based on experimental results Schlinger limited the validity of the expression to the region where y_b is less than 0.7y. Outside this region it is assumed to be constant. Equation (31) is used in the fluid element rigidity matrix $[K_e]$ required in Equation (22).

3.4.4 Solution procedure

Turbulent flow is a function of the eddy viscosity and the boundary shear. The boundary shear is also a function of the velocity distribution. To find the solution it is necessary to assume one of the parameters and to use an iteration procedure. For the obstructed part of the cross-section I calculated the initial velocity with Equation (17). For the unobstructed part I took the average velocity initially. For all the boundary nodes the shear stress was assumed for the first iteration to be equal to the average shear ($\rho \cdot g \cdot R \cdot S$). The velocity distribution between the solid boundary and the edge of the finite element nodes was calculated (Fig. 15). Then the velocities for all other nodes of the finite element network were solved using the Crout reduction procedure (Eq. 23). A new boundary shear was calculated (Eq. 28) for this velocity distribution, for use in the next iteration.

I compared the average velocity, calculated and measured for concrete lined rectangular and trapezoidal sections (QUERNER, 1981). The average velocities agreed within 2% and for most sections were within 1%. The measured and the calculated velocity distributions for a parabolic section are shown in Figure 16. The presence of secondary flow causes the actual maximum velocity to occur below the water surface at midchannel. The present method ignores the secondary flow, which results in the maximum velocity occurring at the water surface. A difference in the magnitude of the velocity is also noticeable in the shallow region of the parabolic section. In other regions a good correlation of results can be observed.

In Section 7.2.2 the method outlined in this section is used to estimate the velocity distribution and flow resistance for different cross-sections.

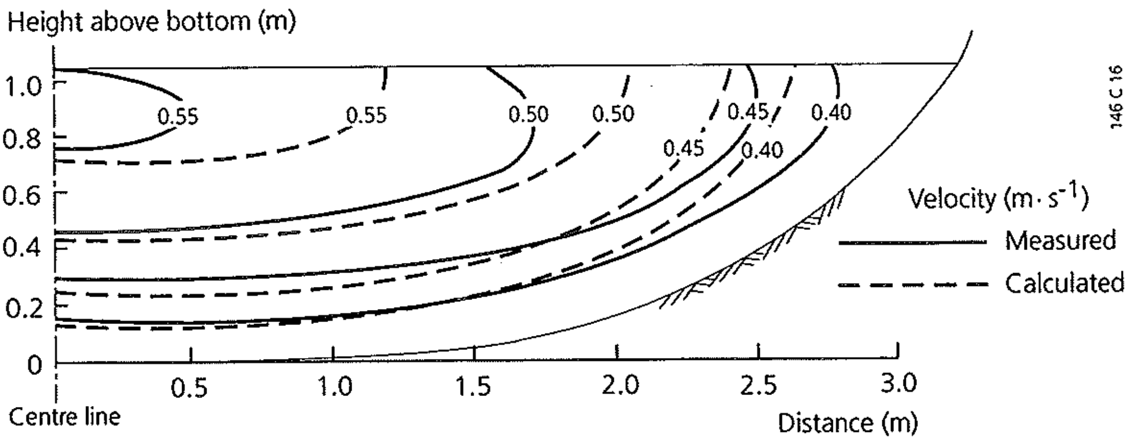


Fig. 16 Velocity distribution calculated and measured in a parabolic section (QUERNER, 1981).

4 Modelling the regional hydrology

4.1 Introduction

The systems approach to hydrological investigations involves measuring observable variables in the hydrological cycle and developing explicit relationships between these parameters (FREEZE, 1971). Numerical models of a regional hydrological system can be derived from physically-based mathematical methods, or by parametric or stochastic methods of system investigation. One can differentiate between a lumped-system model in which the watershed is treated as a 'black box' and a physically based distributed-system model in which the internal processes of the model are analysed. The hydrological cycle is a dynamic system that operates within a set of constraints and physical laws that control the movement, storage and disposition of water within the system. It is a closed system and conforms to the principle of conservation of mass. The advantages of physically based models were discussed in Section 2.4.

Hydrological models have proliferated, as evidenced by the content of IGWMC newsletters and a survey of VOLP and LAMBRECHTS (1988). Generally the models can be classified according to the hydrological subsystem they consider: saturated groundwater models, unsaturated zone models and surface water models. The unsaturated zone models are often used in relation with the prevailing crop system. Advances in computer technology and the reduction in computational time have made it possible to integrate the subsystems into hydrological response models. The pioneering work done by Freeze (FREEZE and HARLAN, 1969) on the integrated modelling in a physically based approach deserves mention. In the last twenty years the need for integrated models to solve many practical problems has become clear.

Various models consider the integration of groundwater (saturated and unsaturated zones) and surface water. They include PREDIS, GELGAM, SHE and SIMPRO. The PREDIS model (GILDING and WESSELING, 1983) is physically based and its structure resembles the SHE model discussed below. The GELGAM model (DE LAAT, 1980) is a quasi three-dimensional model for saturated groundwater flow using the finite difference approach. The unsaturated zone is present in each node of the difference scheme and has one form of land use. The flow is considered to be in a pseudo steady state. The standard version uses the surface water levels as a boundary condition, but at some stage a surface water routing module was attached. The hydrological model SHE (ABBOTT et al., 1986) has been developed jointly by the Danish Hydraulic Institute, The British Institute of Hydrology and

SOGREAH (France). It is intended to provide solutions in water resources studies, where problems arising from conjunctive use of water need to be solved. The SHE model is a physically-based distributed catchment modelling system. The catchment is represented horizontally by an orthogonal grid network for saturated groundwater flow and vertically by a column of horizontal layers at each grid square for the unsaturated zone. The surface water system can be present on the boundaries of the grid squares. The primary components of the hydrological cycle are modelled - snow melt, canopy interception of rainfall, evapotranspiration, overland flow, channel flow, flow in the unsaturated zone and flow in the saturated zone. Each hydrological component in the SHE model is controlled from a central control unit. This enables time-varying components to be used in one system, for instance the time step of the saturated groundwater flow (having time steps in the order of days) can be used in conjunction with the unsaturated zone (having time steps in the order of hours). The SHE model has been applied to the Hupselse Beek area in the Netherlands for four events (period 1977-1978), each lasting between 4-9 days (LUMADJENG and GARDNER, 1989). The model was calibrated with the measured hydrograph, but several combinations of different parameters were possible to simulate this hydrograph. Problems were encountered in running the model with the initial input data. The results also show a remarkable amount of overland flow, which was not found in the field. The interaction between groundwater and surface water in SHE is modelled as one system, being the larger water courses. Therefore the artificial (subsurface) drainage system and the small water courses could not be included in the model. Another possible reason for the poor results might be that the grid system is very coarse (250x250m) and the elevation in the area varies by several metres. In general it was concluded that the outflow hydrograph could be simulated satisfactorily, but the simulated response of the groundwater system was poor.

The SIMPRO model (QUERNER, 1986a) was a pioneering attempt to link a surface water model with a relatively simple one-dimensional model of both the saturated and unsaturated zones. This schematization implies that the regional groundwater flow component is not considered in the model, but included as a boundary condition. In the surface water module an option was included to trace the water within a network of water courses (QUERNER, 1990). In an application this aspect was used to estimate to what extent water from outside the area will flow into the system during dry periods (HENDRIKS, 1990). This kind of information is often very important for nature areas located within agricultural areas.

It can be concluded that some models were very ambitious in trying to model the hydrological components in great detail. Partial success was achieved with the SHE model. However, the modelling system becomes very complicated, needing experts to handle it and requiring much data and extensive computational time. At present such a huge model is not practical to simulate the hydrology over long periods or for large regions showing local variations in geohydrological parameters and land use. To overcome these problems I opted for a simpler approach, especially for the unsaturated zone. The important criteria to be considered in the development stage of the various modules were discussed in Section 2.2. The groundwater module is described in Section 4.3 and the surface water module in Section 4.4. The link between the two modules is described in Section 4.5. In Chapter 5 the verification of the hydrological model on the Hupselse Beek area shows the general possibilities and the shortcomings of this modelling approach.

4.2 The combined surface and groundwater flow model MOGROW

4.2.1 Schematization of the surface and groundwater flow

The general structure of a hydrological model consists of storage elements and transmission links. The configuration of such components either in parallel or in series, can lead to a hydrological model for groundwater and surface water. A 'blueprint' of such a hydrological modelling system was first given by FREEZE and HARLAN (1969).

In recent years I developed for parts of the hydrological system computer models. Each model describes a certain aspect of the regional hydrological system (Table 2).

Table 2 Overview of hydrological processes considered in the regional models (marked x).

Model	Saturated zone			Unsat. zone	Surface water		
	regional (multi-layer)		phreatic level	pseudo steady state	constant water level	unsteady state	
	steady	unsteady				reservoir	network
FEMSATS	x				x		
FEMSAT		x			x		
SIMGRO		x		x		x	
SIMFLOW			x	x	x		
SIMWAT							x
SIMPRO*			x	x			x
MOGROW**		x		x			x

* SIMPRO = SIMWAT + SIMFLOW

** MOGROW = SIMWAT + SIMGRO

The result is a package of models with similar input data and the results can be presented in a graphical form. The hydrological model required for the problem of aquatic weed control considered in this research consists of the regional groundwater flow model SIMGRO and the surface water flow model SIMWAT (Table 2). Combining these models results in the MOGROW model. The processes considered in this model are outlined in Figure 17.

The groundwater flow model, SIMGRO (SIMulation of GROundwater flow and surface water levels), was developed to simulate regional groundwater flow in relation to drainage, water supply, sprinkling, subsurface irrigation and water level

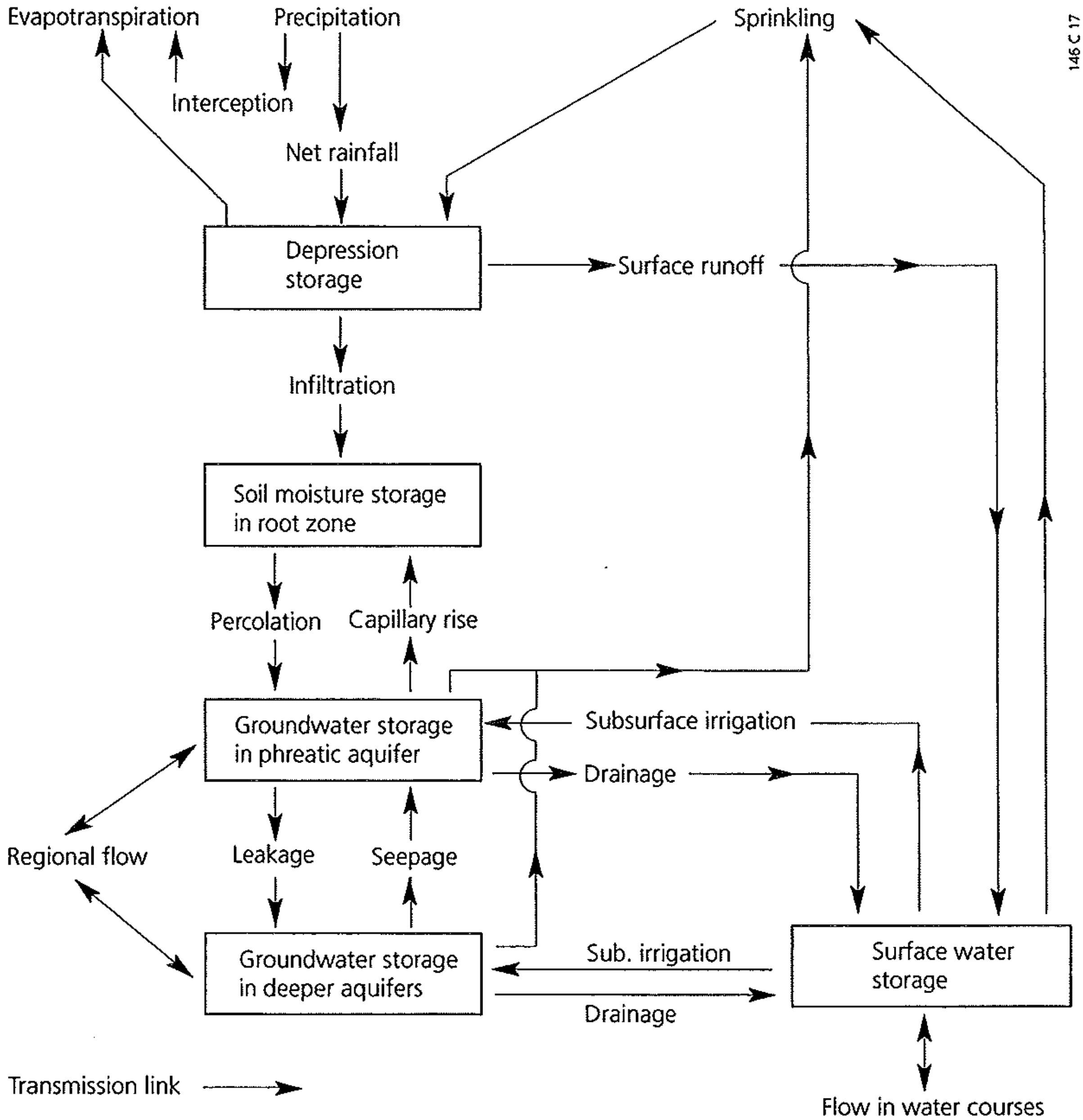


Fig. 17 Schematization of the surface and groundwater flow in the model MOGROW, by means of transmission links and storage elements.

control (QUERNER and VAN BAKEL, 1989). To do this the computer programme FEMSAT (VAN BAKEL, 1978) was extended to include the unsaturated zone and surface water system (Fig. 18). A user's manual is available (QUERNER, 1992a). The model has been used in numerous hydrological studies (VAN BAKEL, 1988; QUERNER, 1988; LANDINRICHTINGSDIENST, 1991; KORTLEVEN and PEERBOOM, 1990; VAN WALSUM, 1992).

The surface water model SIMWAT (SIMulation of flow in surface WATer networks) was developed to describe the water movement in a network of water courses. It also includes special structures such as weirs, pumps, culverts, gates and inlets, necessary for the proper modelling of all water movements within a certain region. The model was set up so that it could be integrated with a groundwater model. Another important criterion for the model was a fairly simple calculation scheme, so that the overall modelling system is not too complicated, is easy to use and does not require vast amounts of input data. A user's manual is available for SIMWAT (QUERNER, 1991) and the model has been used in some studies (HENDRIKS, 1990; QUERNER, 1990).

The combined surface and groundwater model obtained by linking SIMGRO and SIMWAT is called MOGROW (MOdelling GROundwater flow and the flow in surface Water systems).

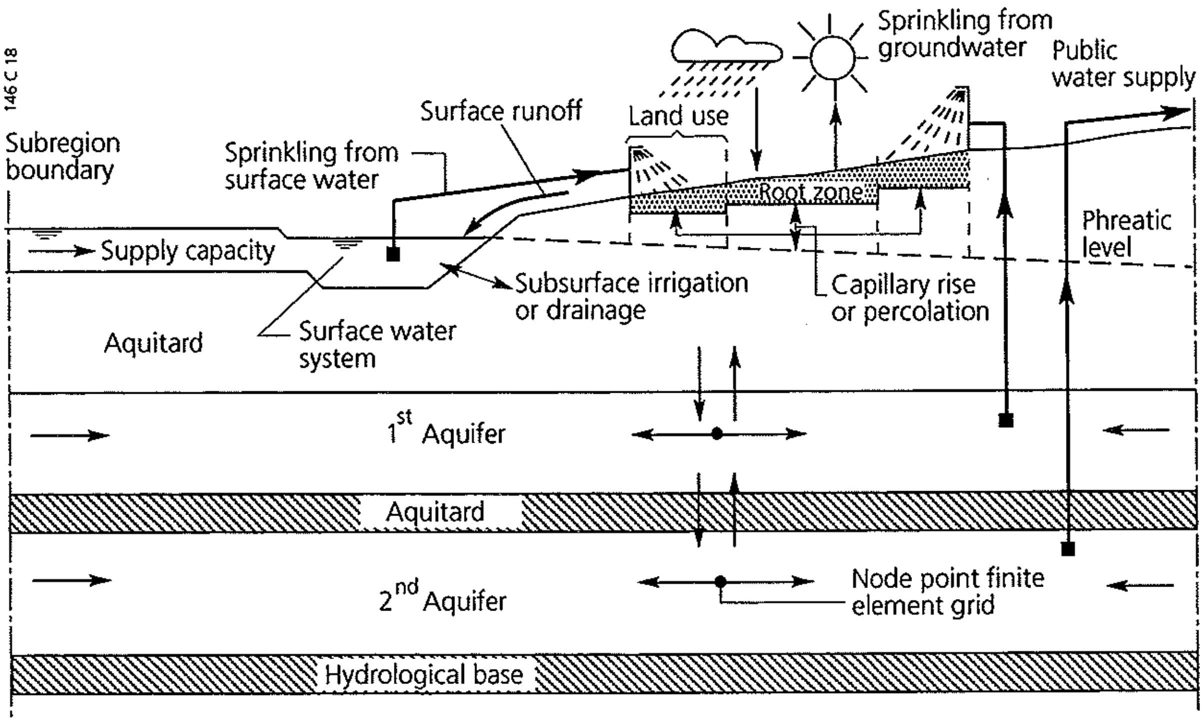


Fig. 18 Schematization in SIMGRO of the hydrological system within a subregion by means of an integration of saturated zone, unsaturated zone and the surface water (QUERNER and VAN BAKEL, 1989).

4.2.2 The theory underlying MOGROW

The hydrological modelling, based on physical laws or approaches will be briefly discussed below. The equations for open channel flow were discussed in Section 3.1. A vast amount of literature is available on the flow of water in the saturated and the unsaturated zones (e.g. BEAR and VERRUIJT, 1987; FEDDES et al., 1988). Here only the basic aspects necessary to understand the actual processes implemented in the computer models as discussed in Section 4.3 are given.

The equation of motion for groundwater flow can be obtained by considering an aquifer layer and applying the principle of linear resistance (Darcy's law) and conservation of mass. The equation describing flow in an aquifer is:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) = A_e \mu \frac{\Delta h}{\Delta t_g} + Q_t \qquad (32)$$

where:

h = hydraulic head (m)

T = transmissivity ($m^2 \cdot d^{-1}$)

x,y = horizontal plane coordinate system (m)

A_e = area (m^2)

μ = storage coefficient (-)

Q_t = constant fluxes such as drainage q_w, capillary rise q_c and extractions for water supply or sprinkling ($m^3 \cdot d^{-1}$)

Δt_g = time step (d)

Note that Equation (32) does not directly contain information about the position of the base or the top of the aquifer. Consequently the calculated value of the groundwater head may lie above the top of an unconfined aquifer or below the base. This means that the transmissivity must be calculated for such an aquifer as a function of the vertical distance from the base to the hydraulic head or groundwater level.

The interaction between surface and groundwater can be described as (ERNST, 1962 and 1978):

$$q_w = \frac{(h_i - h_w^s)}{\alpha_s \Upsilon_s} \qquad (33)$$

where:

q_w = drainage rate ($m \cdot d^{-1}$)

h_i = groundwater level (m)

h_w^s = water level or invert level (bed level) of the surface water (m)
α_s = geometry factor depending on the shape of the water table (range 0.65-0.85; ERNST, 1978)
Υ_s = drainage resistance (d)

The geometry factor is necessary when using the average groundwater levels instead of the levels midway between two ditches. The water courses are generally classified in relation to their dimensions. The drainage resistance per class can be derived from the number of ditches per unit area and the geohydrological properties of the soil. The resistance can be generally regarded as the sum of the vertical, horizontal, radial and entry resistance (ERNST, 1978). The vertical and entry resistance must be measured in the field. The horizontal and radial resistance can be estimated respectively as:

$$\Upsilon_h = \frac{L_w^2}{\sum 8kD} \quad (34)$$

$$\Upsilon_r = \frac{L_w}{\pi k} \ln \frac{4D}{\pi P} \quad (35)$$

where:
L_w = distance between water courses (m)
k = hydraulic conductivity ($m \cdot d^{-1}$)
D = thickness of aquifer (m)
P = wetted perimeter (m)

Part of the drainage resistance as required in Equation (33) is a component for the horizontal flow and can be calculated by Equation (34). For large distances between water courses the horizontal resistance is mostly included in the saturated groundwater flow as shown in Figure 19a. If the distance between nodal points is greater than the distance between water courses, then the horizontal resistance must be included in total in the drainage resistance (Fig. 19b).

The flow of water in the unsaturated zone is commonly described one-dimensionally, using the differential equation proposed by RICHARDS (1931). Also, transient flow can be approximated by series of steady state situations (pseudo-steady state). In this way the unsaturated zone is considered as two reservoirs, one for the root zone and one for the subsoil, being the unsaturated zone between root zone and phreatic level (RIJTEMA, 1971; DE LAAT, 1980). The reservoir for the root zone simulates the storage of moisture in it with inflows and outflows such as precipitation, evapotranspiration, and upward flow or percolation (Fig. 20). Steady state soil

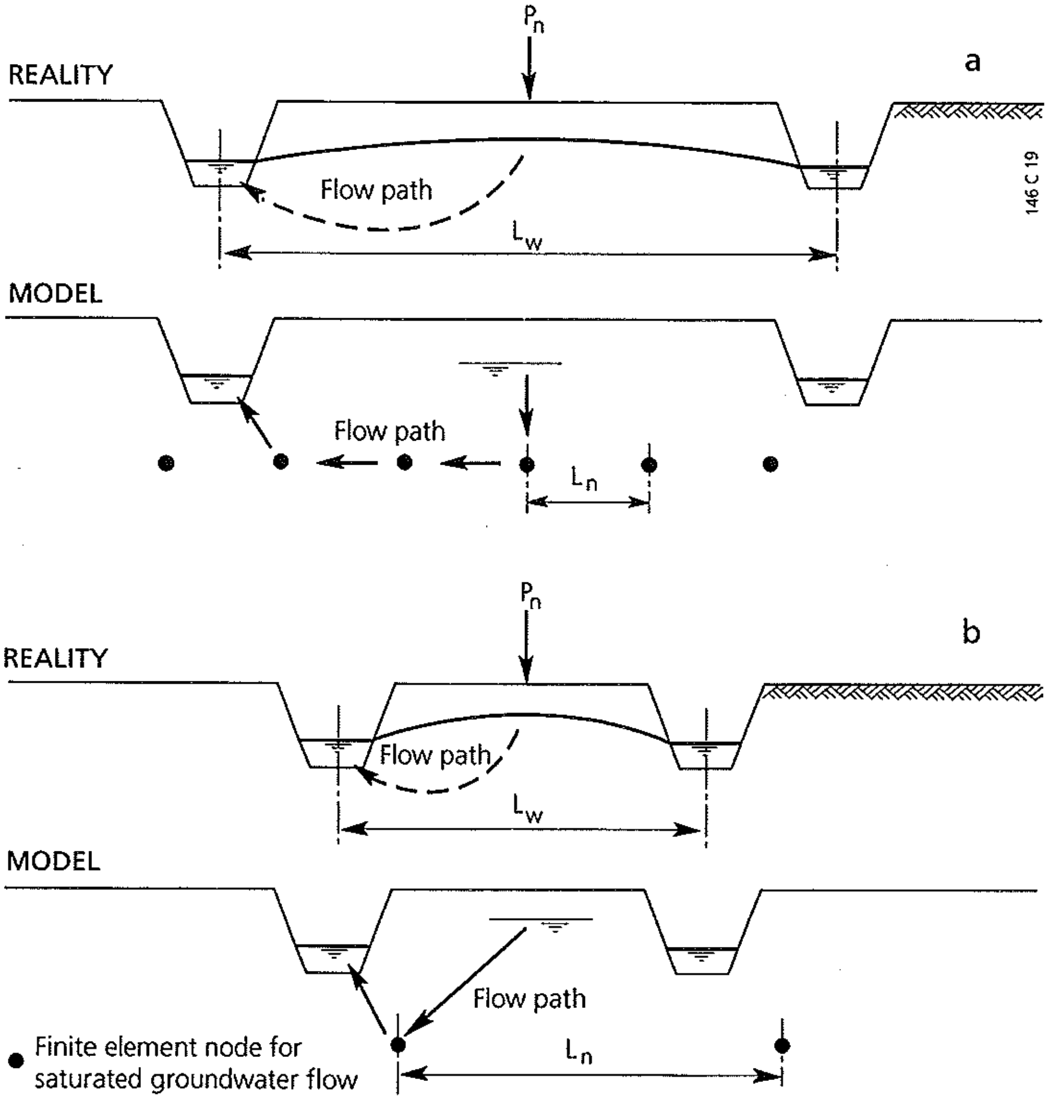


Fig. 19 *Horizontal drainage resistance to be used for the interaction between groundwater and surface water in relation to the spacing of nodes (P_n=net precipitation; L_w=distance between water courses and L_n=distance between nodes).*
a) Nodes are so close that horizontal resistance is part of the saturated groundwater flow
b) Spacing of nodes is larger than distance between water courses, therefore horizontal resistance must be included in the drainage resistance

moisture flow below the root zone can be described by Darcy's equation as:

$$q_c = -k(h_p)(\frac{dh_p}{dz} + 1) \tag{36}$$

where:

q_c = upward flux (m·d^{-1})
h_p = pressure head (m)
z = vertical coordinate with origin at soil surface, directed positively upwards (m)

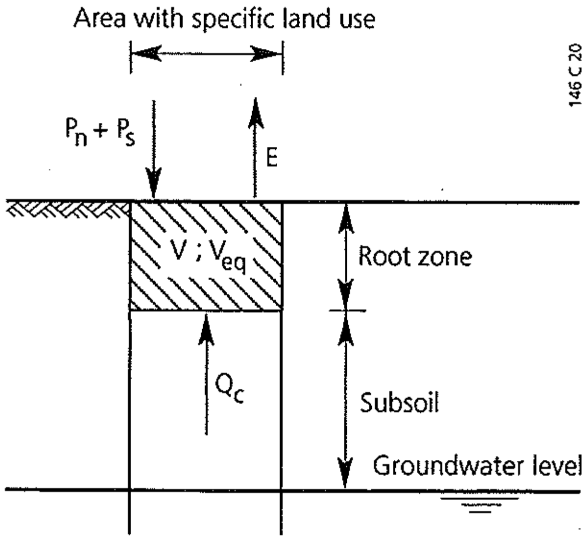


Fig. 20 *Schematization of unsaturated zone per land use in a reservoir for the root zone and one for the subsoil. Nomenclature used: P_n=net precipitation; P_s=sprinkling; E=evapotranspiration; V=moisture storage; V_{eq}=equilibrium moisture storage and Q_c=upward flux.*

Unsaturated zone models often ignore four fundamental aspects, which can sometimes have a significant effect on the soil water movement. They are: surface runoff, perched water tables, hysteresis and preferential flow.

Surface runoff

At the soil surface a number of related processes can be considered. The net rainfall usually infiltrates into the soil. At high intensities it can also result in surface runoff (overland flow) or remain on the surface in depressions (pools of water), as shown in Figure 21. The infiltration of rainfall can increase until a maximum rate is reached. When the actual rainfall intensity exceeds the maximum infiltration rate, the excess water is stored on the surface. The depression storage has a certain maximum capacity. After this maximum capacity is exceeded, surface runoff will occur as well.

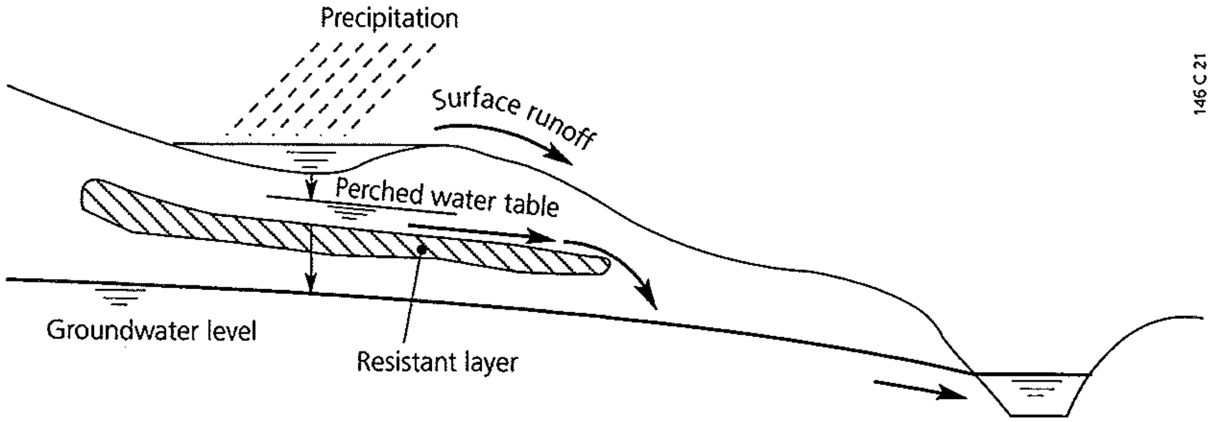


Fig. 21 *The process of surface runoff and perched water tables possibly occurring in reality and the resulting flow of water.*

Perched water tables

A compacted subsoil layer can sometimes occur at some distance below the soil surface in the unsaturated zone. Infiltration through this layer may be very slow, so water tends to accumulate above this layer and a so-called 'perched' water table is formed (Fig. 21). The occurrence of perched water tables above the main groundwater zone has been ignored in most hydrological models. Little information and research about this topic is available. Perched water tables occur only under certain meteorological (rainfall) conditions and show a discontinuous behaviour in time.

Hysteresis

The aspect of hysteresis is caused by pore-space geometry, the presence of entrapped air, shrinking and swelling, and thermal gradients (BEAR and BACHMAT, 1990). As an example the soil moisture characteristics between wet and dry conditions (history of drainage) is shown in Figure 22. For the drying curve (Fig. 22), the air entering is blocked by water and for the wetting curve the water entering is blocked by air. The transitions from one curve to the other is achieved by means of so-called scanning curves (MUALEM, 1974).

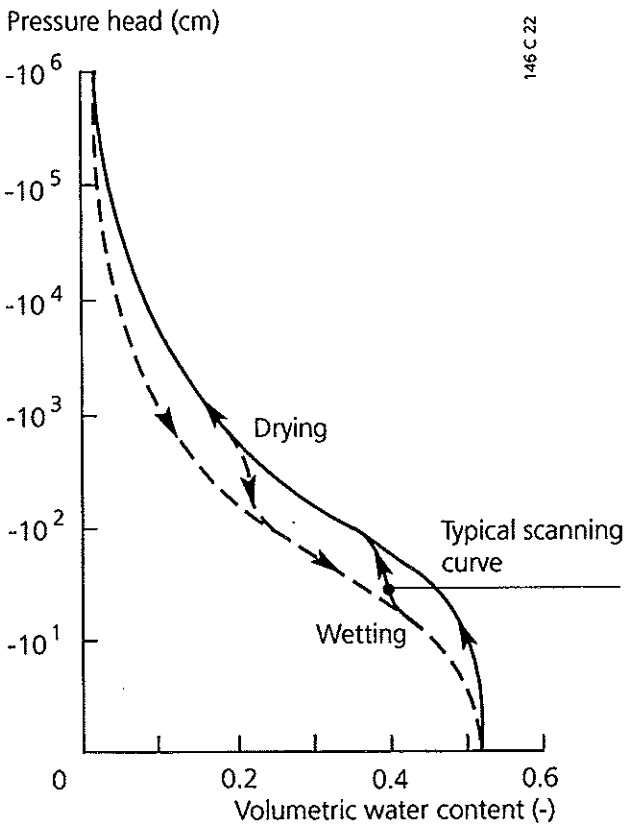


Fig. 22 Relationship between pressure head and volumetric water content, which are different for drying (solid lines) and wetting (dashed lines) conditions, called hysteresis.

Preferential flow

Traditionally the movement of water in the unsaturated zone is assumed to be homogeneous. In reality the movement of water can behave very differently because of varying permeability, cracks, water-repellency of the soil and macro pores. The inhomogeneity may be enhanced by disturbances or influences at or above the ground surface. Examples are rainfall flowing down stems of crops such as maize and the unevenness of the ground surface produced by ploughing and the cultivation of particular crops. Even below a grass cover extreme variations in moisture have been observed over very short distances (BRONSWIJK et al., 1990). The presence of macro pores may be the result of plant roots, the shrinking and cracking of the soil or human interference via tillage. The consequence of preferential flow is that part of the precipitation can reach the groundwater much faster than water flowing through the soil matrix. Dye studies have demonstrated the importance of macro pore preferential flow (VAN OMMEN, 1989; STEENHUIS et al., 1988). The installation of subsurface drains can further emphasize preferential flow paths. This preferential flow can be intercepted by the subsurface drains. Water flowing out of drains even though the groundwater level is well below the drains has been reported by STEENHUIS et al. (1988).

The first simulations with the MOGROW model for the Hupselse Beek area revealed various shortcomings to describe groundwater levels and discharges properly (Section 5.2). Therefore the fundamental processes described above were included to improve the model results. The consequences of doing this are discussed in Section 5.4.

4.3 The groundwater module SIMGRO

4.3.1 Schematization of land use and the hydrological system

To model regional groundwater flow the system has to be schematized geographically, both horizontally and vertically. Land use is schematized in the first aggregation level. The second aggregation level deals with subregions describing moisture vertically in the unsaturated zone. The third level covers various subsurface layers for saturated groundwater flow (Fig. 18).

The model considers subregions to be subdivided into different land uses. The area of each type of land use should be known as a percentage of the subregion, but not its geometrical position. The total area of a certain land use may represent many patches scattered over a subregion. I defined four main categories of land use that are important for the calculations of the various water movements: agricultural areas, urban areas, nature reserves and woodland. The differentiation of the agricultural area should take into account all major categories of land use that play a role in the hydrological cycle. Important aspects in this respect are the difference in evapotranspiration, rooting depth and the irrigation by means of sprinkling. The urban areas are divided into areas with an impermeable surface (e.g. houses, streets) and a permeable surface (gardens and parks). In the Netherlands the sewage system generally also collects all water from the impermeable surface. In the model this sewer system is treated as a reservoir with inflow and extractions. Nature reserves often have a typical vegetation of grass under wet or dry conditions, as does woodland. Woodlands are distinguished because they have quite different evapotranspiration values and thicknesses of the root zone.

The groundwater system is schematized into a number of layers, being horizontal flow in water-bearing layers (aquifers) and vertical flow in less permeable layers (aquitards). The aquifer is unconfined when it has a free water surface or confined when it is enclosed above and below by aquitards. Geohydrological information such as hydraulic transmissivity, vertical resistance, layer thickness, storage coefficient, is required for each layer. Boundary conditions can be in the form of a flux or hydraulic head. The region to be modelled is subdivided into finite elements. In this way it is easy to describe complex geometrical configurations with relatively simple element shapes.

The often dense network of water courses, related to size and density, is important for the interaction between surface and groundwater. The smaller water courses I assumed to be spread evenly over a finite element or subregion. These are primarily involved in the interaction between surface water and groundwater: commonly called the secondary water courses, the tertiary water courses and the trenches (Fig. 23). A so-called channel system can be present as well, but in specific nodes. It should represent the larger channels, also considered in the surface water model. In this way four drainage subsystems can be used in the model for the interaction between surface and groundwater.

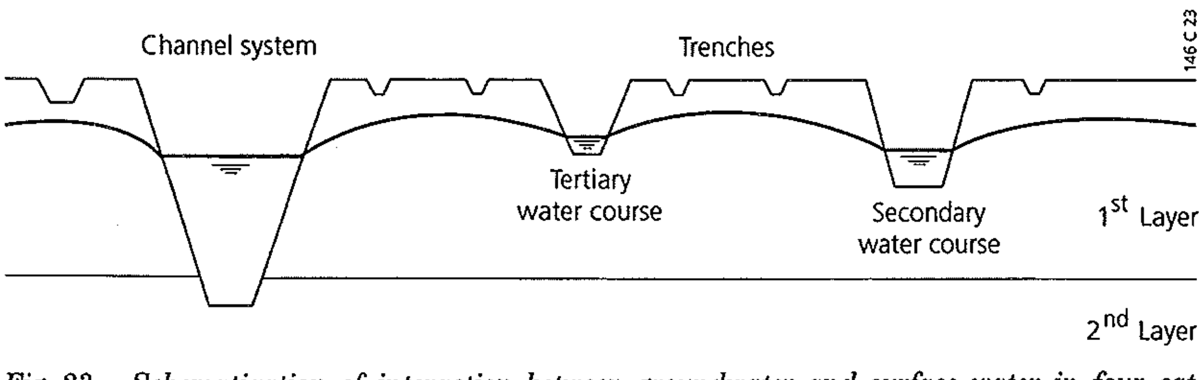


Fig. 23 *Schematization of interaction between groundwater and surface water in four categories. The channel system interacts with different layers of the saturated zone.*

Ideally the flow and retention of water in the unsaturated zone should be calculated for each nodal point and per land use separately, because the parameters involved in modelling the water flow in the unsaturated zone are very non-linear. To take all specific relations and different flow behaviour in the root zone into account would require per nodal point and per land use a model to simulate the unsaturated zone. This would require a great amount of input data and a heavy demand on both computer time and storage. Therefore I introduced a simplification: per subregion and per land use one model (reservoir) is used to calculate moisture storage, evapotranspiration and capillary rise (or percolation), using average hydrological conditions. This implies that subregions have homogeneous conditions with respect to soil type and water table fluctuations. The water table fluctuations, characterized by minimum and maximum groundwater levels over the years, should not vary considerably within a subregion. The average depth of the groundwater level below soil surface is used to calculate the characteristics of the unsaturated zone (upward flux, storage coefficient and equilibrium moisture coefficient).

4.3.2 The saturated zone

SIMGRO uses the finite element method as a basis to describe the regional saturated groundwater flow. In the finite element process the displacements or flux terms are usually specified by functions per element, each nodal parameter influencing only adjacent elements. The region is thus an assemblage of individual elements that are either triangular (Fig. 24) or quadrilateral in shape. The finite element method describes the head at every point (x,y) of an element by means of a

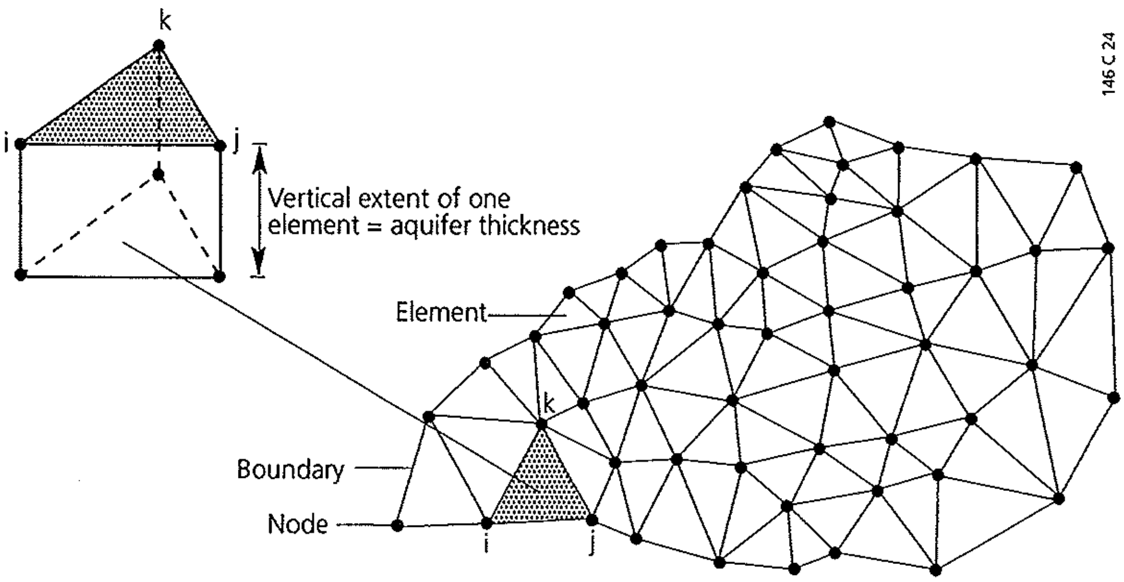


Fig. 24 Subdivision of solution domain into finite triangular elements. Each element extends over the layer thickness and has nodes i, j and k numbered counter clockwise.

linear interpolating function. For further details on the finite element method applied to groundwater flow see for instance WANG and ANDERSON (1982) or NEUMAN (1973).

Unsteady flow conditions will indicate that during a time interval from t to $t+\Delta t_g$ a net quantity of water will flow to or from a node i. The amount of water involved will result in a rise or fall of the hydraulic head. Therefore using Equation (32) the continuity equation per node and layer can be written as:

$$A_i \mu \frac{\Delta h}{\Delta t_g} = W[\sum_j Q_{ji} + Q_a]^{t+\Delta t_g} + (1-W)[\sum_j Q_{ji} + Q_a]^t \tag{37}$$

where:

A_i = area of node i (m^2)
μ = storage coefficient unsaturated zone as shown in Fig. 25 (-)
Δh = change in hydraulic head over time step (m)
W = weighting parameter between the time levels t and $t+\Delta t_g$ (-)
Q_{ji} = flow from adjacent nodes j to node i ($m^3 \cdot d^{-1}$)
Q_a = all fluxes from adjacent layers, the unsaturated zone, interactions with the surface water and extractions ($m^3 \cdot d^{-1}$)

A weighting parameter W of 0.5 means a straightforward averaging between two time levels (Crank-Nicolson approximation). For the top layer the storage coefficient μ is the storage of water in the unsaturated zone from the underside of the root zone down to the phreatic level (Fig. 25). This means that the storage coefficient varies with the depth of the groundwater level below soil surface. Therefore the storage coefficient must be integrated over the variation in groundwater level for the time step (Δh). The storage coefficient is therefore updated within the iteration process. As will be discussed in Section 4.3.4, when a perched water table is present the storage coefficient for the saturated zone module is from the top of the resistant layer down to the phreatic level (μ_p in Fig. 27). For confined aquifers the storage coefficient to be used in Equation (37) is the elastic storage due to the change in hydraulic head. Assuming further that:

$$Q^{t+\Delta t_g} = Q^t + \Delta Q \tag{38}$$

and

$$h^{t+\Delta t_g} = h^t + \Delta h \tag{39}$$

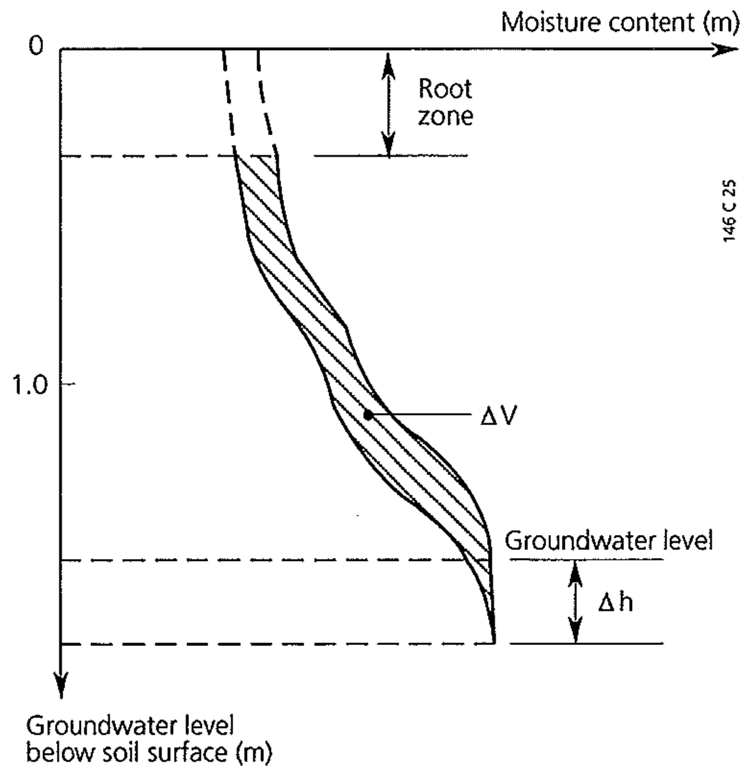


Fig. 25 *The storage coefficient μ is calculated as the change in moisture storage in the unsaturated zone below the root zone (ΔV) divided by a change in groundwater level (Δh).*

Equation (37) can be rewritten as:

$$A_i \mu \frac{\Delta h}{\Delta t_g} = \sum_j Q_{ji}^t + Q_l^t + W[\sum_j \Delta Q_{ji} + \frac{dQ_l}{dh} \Delta h] + Q_t \quad (40)$$

where:

Q_l = boundary flow dependent on the groundwater level (leakage and drainage to channel system) ($m^3 \cdot d^{-1}$)

Q_t = constant fluxes over time step such as drainage to the secondary and tertiary systems, capillary rise and extractions for water supply or sprinkling ($m^3 \cdot d^{-1}$)

The first two terms on the right-hand side of Equation (40) represent the flows to or from node i at time t and the third and fourth terms are the actual change in flow over the time step considered. It can be regarded as an implicit calculation scheme (for saturated groundwater flow) in which the time step can be chosen freely. The interaction with the unsaturated zone (Section 4.3.4) and the surface water (Section 4.5) are treated explicitly. Equation (40) is solved by the Gauss-Seidel iterative method, using an over-relaxation factor as given by Carre (REMSON et al., 1971).

In an aquifer the flow over the sides of a finite element are represented by the flows between nodes as (QUERNER and VAN BAKEL, 1989):

$$Q_{ji} = T^t_{ij}(h_j - h_i) \tag{41}$$

where:

T_{ij} = transmissivity matrix for horizontal flow (for a phreatic aquifer dependent on the water-bearing part of the layer) ($m^2 \cdot d^{-1}$)

The flow between two nodes is linearly related to the difference in hydraulic head. Given the changes in hydraulic head between two adjacent nodes, Equation (41) can also be used to define the change in flow over the time step. The external flow Q_l and the constant flux Q_t are composed of the following flux terms:

$$Q_l = -A_i q_l - L_i q_s \tag{42}$$

$$Q_t = -A_i(q_c + q_e + \sum_p q_w) \tag{43}$$

where:

A_i = area of the node (m^2)
q_l = leakage to other layers ($m \cdot d^{-1}$)
L_i = channel length (m)
q_s = drainage to channel system per unit length ($m^2 \cdot d^{-1}$)
q_c = upward flow ($m \cdot d^{-1}$)
q_e = extractions for public water supply as well as for sprinkling ($m \cdot d^{-1}$)
q_w = drainage to secondary and tertiary water courses ($m \cdot d^{-1}$)
p = number of summations of interaction groundwater and surface water, being $\Delta t_g / \Delta t_s$

For the channel system the drainage is calculated per unit length using Equation (33) and a corresponding drainage resistance. A time step Δt_s is used to calculate the water levels in the surface water system. In general it is smaller than the time step for the groundwater system. The drainage for a time step of the groundwater system is computed as the sum of the drainage for the time steps of the surface water submodel (see also Section 4.5).

The boundary conditions for the aquifers can be either a prescribed head (Dirichlet condition), which means the head is specified in time, or a prescribed flux is given in time (Neuman condition). The flow across a boundary is divided up evenly between adjacent boundary nodes.

4.3.3 Drainage

The ditches that are actually involved in the interaction between surface water and groundwater are commonly the secondary and tertiary water courses, together with the shallow trenches. In addition, a so-called channel system may be present (see also Fig. 23). Using Equation (33) the interaction is calculated for each of these subsystems. The secondary and tertiary water courses are assumed to be spread evenly over the area of a node and the interaction with the groundwater system is only with the top layer of the saturated zone. The channel system can be present too, but only in specific nodes. This system is also considered as the network of water courses in the surface water model SIMWAT (Section 4.4).

4.3.4 Modelling soil moisture in the unsaturated zone

The unsaturated zone is considered by means of two reservoirs, one for the root zone and one for the subsoil (Fig. 20). If the equilibrium moisture storage for the root zone is exceeded, the excess water will percolate to the saturated zone. If the moisture storage is less than the equilibrium moisture storage, then the result will be an upward flow from the saturated zone. Here the equilibrium moisture storage is defined as the amount of moisture corresponding with a steady-state situation where the flow is zero. The height of the phreatic surface is calculated from the water balance of the subsoil, using a storage coefficient which is dependent on the depth of the groundwater level below soil surface (Eq. 40). As stated in Section 4.3.1, ideally the flow and retention of water in the unsaturated zone should be calculated for each nodal point and per land use separately. The simplification introduced is that per subregion and land use one model (reservoir) is applied for calculating moisture storage, evapotranspiration and capillary rise/percolation. In this case average hydrological conditions over the subregion are used. For example the amount of capillary rise in a subregion is now dependent on the average depth of the groundwater.

The root zone for which the water balance is considered has a thickness d_r, which is a function of the land use j and the physical soil unit s. Therefore:

$$d_r = f\{j, s\} \tag{44}$$

In the model a constant thickness of the root zone has been assumed all year round, with no changes during and over the years.

For the root zone of each land use with area A_j the change of moisture storage ΔV over a time step Δt_g is:

$$\Delta V = (P_n + P_s - E)\Delta t_g \tag{45}$$

where:
P_n = net precipitation, being precipitation minus interception (m)
P_s = rainfall from sprinkling (m)
E = evapotranspiration (m)

The rainfall is corrected for plant interception, dependent on the type of crop. The rather complex process of surface runoff as discussed in Section 4.2.2 is simplified in the SIMGRO model by a reservoir (depression storage) as shown in Figure 26. The net rainfall is stored in this reservoir with the infiltration occurring at the bottom. The infiltration rate has an upper limit. When the depression reservoir is full the excess is treated as surface runoff. Due to irregularity in sprinkling (e.g. wind, overlap) it has been assumed that 10% of the sprinkling is not stored in the root zone, but percolates directly to the saturated zone. Irrigation by means of sprinkling and the evapotranspiration depends on the crop and moisture storage in the root zone (Section 4.3.5).

The moisture storage of the root zone at equilibrium condition V_{eq} is calculated with the function:

$$V_{eq} = f\{s, d_r, h_r^*\} \tag{46}$$

where:
s = physical soil unit (-)
d_r = thickness of root zone (m)
h_r^* = average depth of the groundwater level below soil surface in a subregion (m)

If the moisture storage $V^t + \Delta V$ is above equilibrium condition, percolation occurs, otherwise there is an upward flux. Therefore:

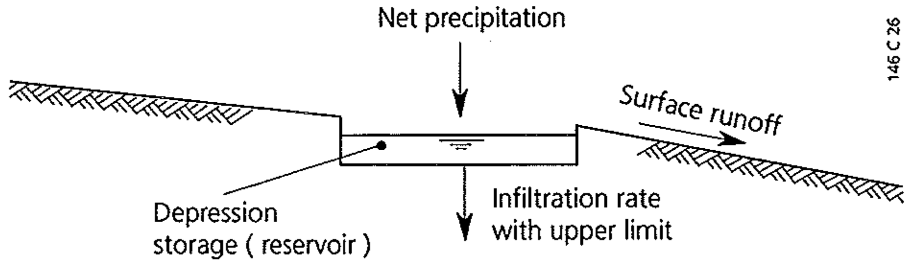


Fig. 26 Calculation of infiltration from net rainfall, using a depression storage and possible surface runoff (net rainfall is precipitation minus interception).

$$q_c = f\{s, h_r^*, d_r, V_{eq}\} \qquad V^t + \Delta V < V_{eq} \qquad \text{for upward flux} \tag{47}$$

or

$$q_c = \frac{V_{eq} - (V^t + \Delta V)}{\Delta t_g} \qquad V^t + \Delta V \geq V_{eq} \qquad \text{for percolation} \tag{48}$$

The moisture storage for the next time step can now be calculated as:

$$V^{t+\Delta t_g} = V^t + \Delta V + \Delta t_g q_c \tag{49}$$

The characteristics of the unsaturated zone, i.e. upward flux, storage coefficient and equilibrium moisture storage of the root zone, are calculated from the moisture retention curve and hydraulic conductivity curve. The CAPSEV model (WESSELING, 1991), based on the assumption of steady state soil moisture flow (Eq. 36), was used to calculate the three parameters. The parameters are required by the model per physical soil unit and for groundwater depths between 0.0 and 2.5 m (interval 0.1 m). Input data for three root zones, being 0.25, 0.50 and 1.0 m in depth, are used for the different land uses.

I verified the model concept with results from a more detailed model of the unsaturated zone, SWATRE. This model is a transient one-dimensional finite difference model for the unsaturated zone with water uptake by roots (FEDDES et al., 1978; BELMANS et al., 1983). The simplified approach in the model SIMGRO has a tendency to yield less evapotranspiration (3-8%) and less capillary rise (QUERNER and VAN BAKEL, 1989) than the model SWATRE. I also compared the results with field data of Hupsel (see Section 5.3).

Perched water tables

There are two possible ways to treat a perched water table in a groundwater flow model. One way is to consider it per land use as part of the unsaturated zone module. In this way the horizontal flow component above the resistant layer is considered at a subregional scale, because the actual crop may be present anywhere within the subregion. Another way is to consider the resistant layer as part of the saturated zone module. The second layer of the schematized saturated zone is now the resistant layer and perched water tables are formed in the first layer. In this way a regional flow component of the perched groundwater is possible per nodal point of the finite element network. The third layer may be partially unsaturated in time or become a confined aquifer, with only an elastic storage as the retention storage. Assuming perched water tables are present within large areas is often unrealistic. Resistant layers are present only locally, resulting in a peak flow rate towards the drainage system. Therefore perched water tables were treated per land use and they were quite simple to integrate in the unsaturated zone module (Fig. 27). In this way the perched water table can be considered as an intermediate reservoir between the root zone and the main water table.

A perched water table will form when the percolation from the root zone (i.e. a negative upward flux q_c) is more than the maximum flux through the resistant layer. The percolation is an inflow in the reservoir, and it must be in equilibrium with the storage, horizontal flow to the surface water system (or drains) and the percolation through the resistant layer (Fig. 27). The water balance for the reservoir to represent the perched water table can be written as (per unit area of land use):

$$\mu \frac{\Delta h_t}{\Delta t_g} = -q_c - q_a - q_i \tag{50}$$

where:

μ = storage coefficient below root zone until the perched water table (Fig. 27) (-)
Δh_t = change in perched water table over time step (m)
q_c = upward flux to root zone $(m \cdot d^{-1})$
q_a = horizontal flow above the resistant layer to the surface water system $(m \cdot d^{-1})$
q_i = percolation through resistant layer $(m \cdot d^{-1})$

The storage coefficient μ represents the change in moisture below the root zone down to the groundwater level (Fig. 25). This storage coefficient can also be used for the perched water table (Fig. 27). When a perched water table occurs, then the storage coefficient μ_p is used in Equation (37), for the unsaturated part below the resistant layer, to calculate the main groundwater table. The horizontal flux towards the surface water system or subsurface drains is calculated with Equation (33). The percolation through the resistant layer is calculated as:

$$q_i = \frac{h_t - (h_k - d_p)}{c} \tag{51}$$

where:

h_t = perched water table head (m)
h_k = top of resistant layer (m)
d_p = thickness of resistant layer (m)
c = vertical resistance of layer d_p (d)

From Equation (50) the new perched water table for the next time step can be written as:

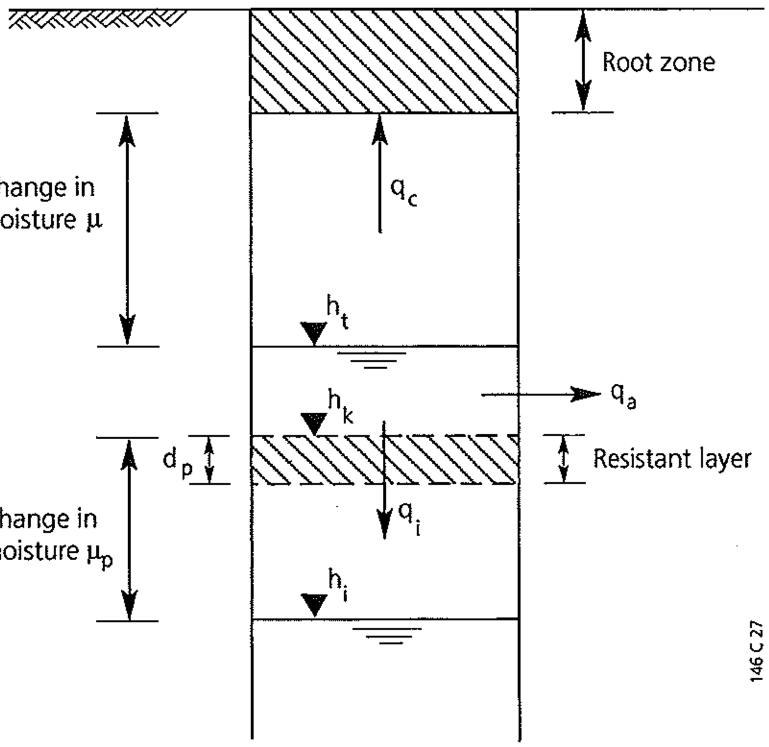


Fig. 27 Schematization of resistant layer and perched water tables per land use in the unsaturated zone. Nomenclature used: q_c=upward flux to root zone; h_t=perched water level; q_a=horizontal flow over resistant layer; h_k=top of resistant layer; d_p=thickness of resistant layer; q_i=percolation through resistant layer; h_i=phreatic level; μ=storage coefficient unsaturated zone (below root zone) and μ_p=storage coefficient unsat. zone below top of resistant layer

$$h_t^{t+\Delta t_g} = h_t^t + \frac{-q_c - q_a - q_i}{\mu/\Delta t_g} \qquad (52)$$

Equation (52) can be solved iteratively. In time the perched water table may disappear, resulting in percolation from the root zone equalling the flux on the main water table. Also, the rising main water table may reach the resistant layer, thereafter the part below the resistance layer is considered to be confined. Then the storage coefficient μ_p (Fig. 27) becomes small and should represent the elastic storage.

Hysteresis

The soil moisture characteristic is generally assumed to yield the drying conditions. In the pseudo-steady state modelling concept used for the unsaturated zone the aspect of hysteresis is only considered for the root zone. The soil moisture curves for drying and wetting have been shown in Figure 22. From these relations the equilibrium moisture storage for the root zone is calculated (Eq. 46). Hysteresis gives a smaller equilibrium moisture storage for wet conditions than for drying conditions, as shown in Figure 28. Hence the root zone, simulated by a reservoir of a certain capacity can change its maximum capacity in time. When a relative low moisture storage (dry situation) coincides with heavy rainfall, hysteresis occurs. The equilibrium moisture storage is then reduced and represented by the line for wet conditions (Fig. 28). After a situation of wetting has occurred, the equilibrium moisture storage is slowly increased to rejoin the curve for drying conditions (assumed increase of 0.25 mm·d^{-1}). For groundwater levels that are the same in time or changing, this will result in trajectories as shown in Figure 28 by means of the dashed lines. In this way an approximation of the hysteresis loops in the soil water functional relationship is achieved.

Preferential flow

Preferential flow is treated by considering two domains. A fraction of the net precipitation, e.g. 10-15%, bypasses the root zone (Fig. 29). The remainder flows into a reservoir considered to represent the root zone. The bypass or preferential flow can either go directly to the groundwater or, if subsurface drains are present, a portion flows directly into the drains (Fig. 29). What portion of the water flows into the drains depends on the depth of the groundwater.

Interaction with saturated zone

For the characteristics of the unsaturated zone certain parameters, such as equilibrium moisture storage and capillary rise, are related to the depth of the groundwater level. Because the unsaturated zone is considered at a subregional scale, the

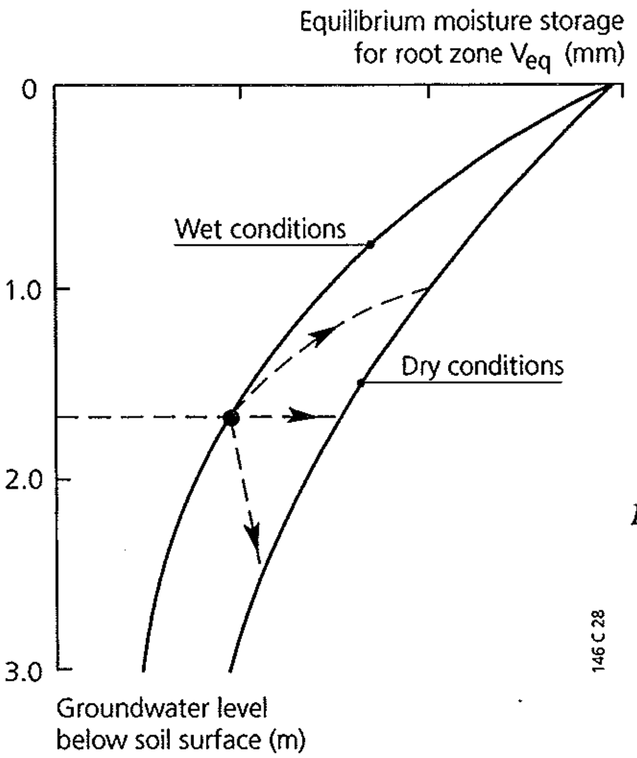


Fig. 28 Equilibrium moisture storage, V_{eq}, for the root zone dependent on the depth of the groundwater level and the effect of hysteresis. The dashed lines represents typical trajectories of groundwater levels and the slow increase in V_{eq} in time after wetting has occurred.

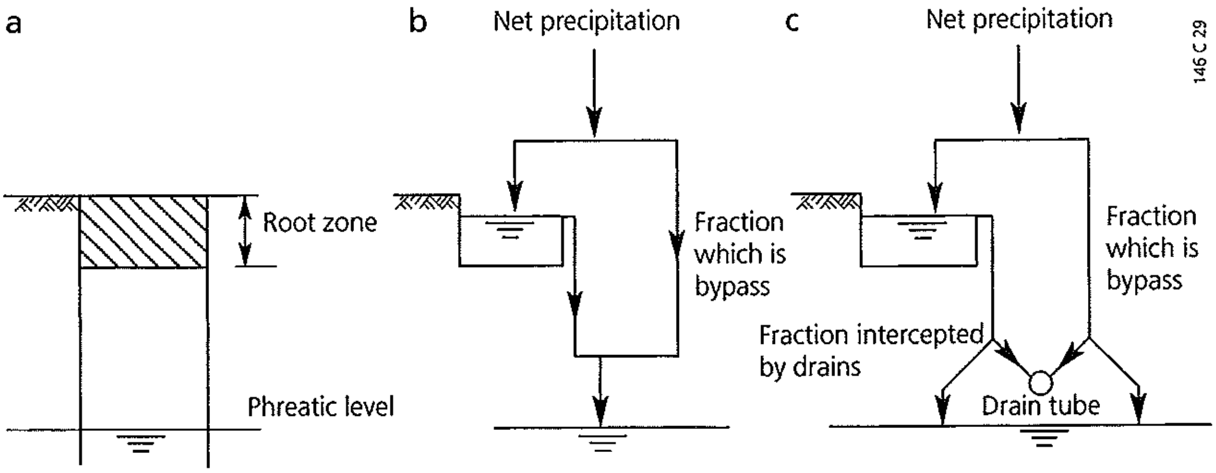


Fig. 29 Schematization of preferential flow, being a fraction of net precipitation going directly to the groundwater or the subsurface drains.
a) Unsaturated zone
b) Flow paths for preferential flow
c) Flow paths for preferential flow when subsurface drains are present

average ground level and the average depth of the groundwater must be known. After the hydraulic head in each node has been calculated with Equations (39) and (40), the average head per subregion is calculated as the weighted average of the area per node.

The extractions from groundwater for sprinkling, and the percolation or capillary rise, are also calculated on the aggregation level of a subregion. Subsequently the fluxes are attributed to the nodes of a subregion by multiplying by the relative areas of the respective nodes.

4.3.5 Evapotranspiration and sprinkling

SIMGRO requires the potential evapotranspiration of a reference vegetation as input data. The potential evapotranspiration for other vegetation types is calculated using crop factors (FEDDES, 1987). The potential evapotranspiration of the reference vegetation is computed from meteorological data using the Makkink equation for grass (DE BRUIN, 1987). The potential evapotranspiration for woodland is calculated as the sum of transpiration and interception. For deciduous forest the leaf area is considered for including soil evaporation as well (QUERNER, 1992b). Soil evaporation is also used for agricultural land, when no crops are present. The actual evapotranspiration E is calculated as:

$$E = \alpha E_p \tag{53}$$

with

$$\alpha = f\{\frac{V}{V_s}\} \tag{54}$$

where:

α = relative evapotranspiration (-)

E_p = potential evapotranspiration ($m \cdot d^{-1}$)

V_s = saturated soil moisture storage in the root zone (m)

The response of some crops to waterlogging can be taken into account. This requires some different functions between relative evapotranspiration and the moisture storage in the root zone (Fig. 30). Figure 30 shows that root water uptake is zero when V/V_s is below 0.05, assumed as the wilting point (FEDDES et al., 1978). Line I assumes no effect of waterlogging. When V/V_s is equal to unity certain plants will have zero root water uptake, which is shown by line II (reduction from 0.95 to 1.0 - Fig. 30). Line III is for plants which are very sensitive to waterlogging (reduction from 0.9 to 0.95 - Fig. 30).

SIMGRO also characterizes the agricultural areas with respect to a water availability condition by means of a crop production level. A high production level means a

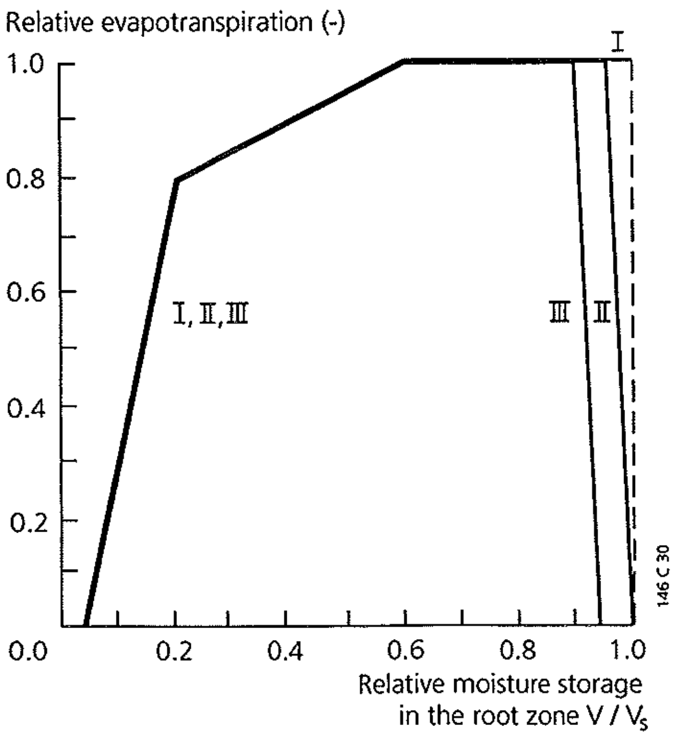


Fig. 30 Relationship for calculating relative evapotranspiration from soil moisture conditions of root zone (for description of lines see text).

higher water demand in the growing season. The demand is met by means of sprinkling, where each production level has its own criterion for applying the sprinkler irrigation in terms of available moisture in the root zone. The sprinkling is continued as long as the soil moisture storage is below a certain level. The model allows for sprinkling to be simulated according to a rotation scheme. This means that within the rotation time, say 7 days, the total area of the specific land use is sprinkled with a certain amount (e.g. 20-25 mm). A loss of 5% is taken into account for evaporation of intercepted water. Due to irregularities in the distribution of sprinkling, 10% of the water is not stored in the root zone, but percolates directly to the saturated zone. The threshold values for applying sprinkling depend on the type of crop and are given as the relative moisture storage in the root zone. The relative moisture storage is defined here as the present moisture storage divided by the equilibrium moisture storage for a depth of the groundwater level of 1.0 m. Case studies carried out with this modelling concept have shown that these assumptions give amounts of sprinkling also found in the field (QUERNER and VAN BAKEL, 1989; HENDRIKS, 1990).

In general the water for sprinkling may be extracted from surface water and/or from groundwater. The maximum extraction from surface water reflects the availability of surface water and the area to be served. Depending on the maximum supply of surface water and the amount of subsurface irrigation, the extraction from the surface water system for sprinkling can be limited or even stopped.

4.3.6 Urban area

Storm water from all impermeable surfaces is collected in the Netherlands in a combined storm water and sewage system. I modelled this sewer system as a reservoir with a certain amount of storage and an extraction, being the pump capacity of the treatment works (Fig. 31). When the maximum capacity of the sewers is reached, the excess is treated in total as an emergency overflow. The effluent from the treatment plant and the emergency overflow are each assigned to a water course of the surface water model.

4.4 The surface water module SIMWAT

In the Netherlands the surface water system is often a dense network of water courses. It is not feasible to explicitly account for all these water courses in a regional computer model. The surface water levels in the major water courses are important for the flow routing and to estimate the drainage or subsurface irriga-

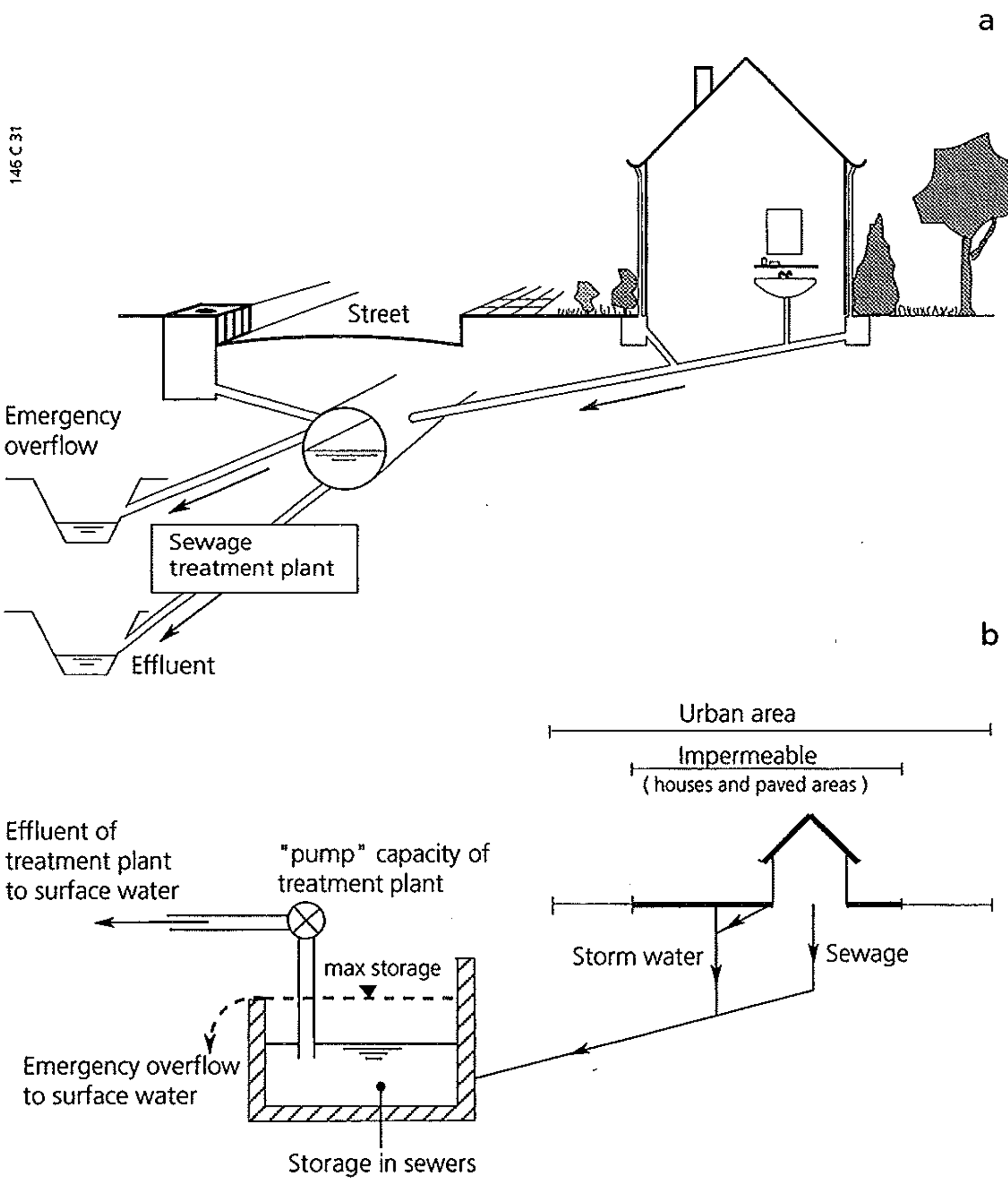


Fig. 31 *Schematization of impermeable surfaces in urban areas and the sewer system collecting the storm water and sewage.*
a) Simplification of reality
b) Schematization of sewage system in the model as a reservoir

tion. Therefore in SIMWAT the major water courses are modelled explicitly as a network of sections, the other water courses are treated as reservoirs and connected to this network (Fig. 32).

From the differential Equations (1) and (2) one can obtain the difference equations by considering a section and applying the principles of conservation of mass and momentum. An amount of water stored by a change in water level, Δh over a storage area S_i can be written as:

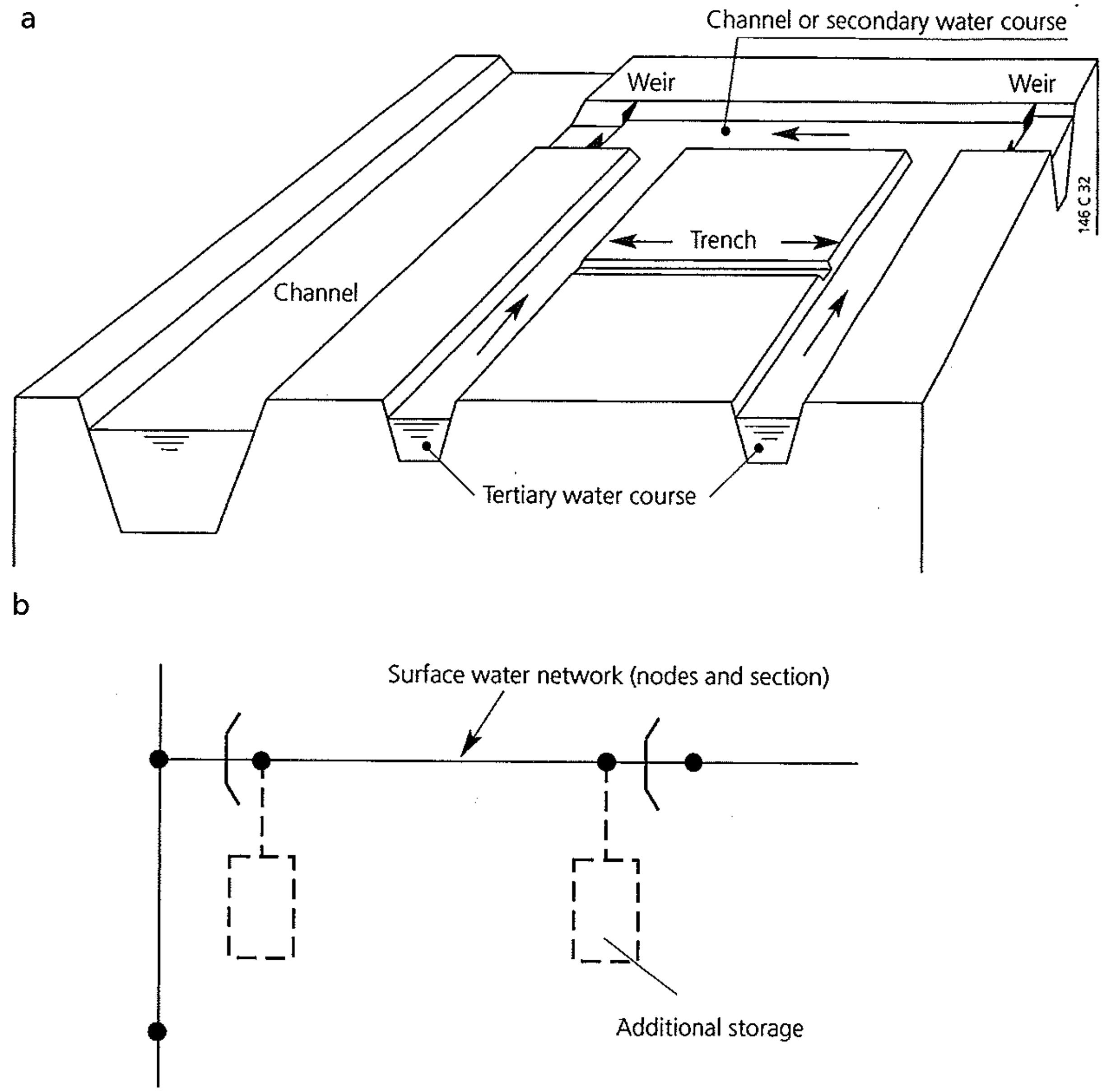


Fig. 32 *Schematization of the surface water system.*
a) Typical layout of surface water system in reality
b) Schematization in the SIMWAT model as sections and nodes for the major water courses and additional off-channel storage reservoirs for all minor water courses

$$S_i \frac{\Delta h}{\Delta t_s} = W[\sum_j Q_{ji} + Q_d]^{t+\Delta t_s} + (1 - W)[\sum_j Q_{ji} + Q_d] \tag{55}$$

where:

S_i = storage area for node i at time t (m^2)
W = weighting parameter for flows between time levels (-)
Q_{ji} = flow from node j to node i ($m^3 \cdot s^{-1}$)
Q_d = external inflow such as drainage ($m^3 \cdot s^{-1}$)

The above equation can be applied to a node as shown in Figure 33. The storage of water is considered to be concentrated in the nodes and the transportation of water occurs in the branches between two nodes. The weighting parameter is necessary to describe the variable hydraulic parameters as a function of time. The closer the weighting parameter is to 0.5 (Crank-Nicolson scheme) the greater is the accuracy but for numerical stability it should be larger than 0.5. Therefore the weighting parameter is generally assigned the value 0.55 (VREUGDENHIL, 1973). Another scheme frequently used is the Preismann scheme, using at every nodal point a water level and a flow rate (CUNGE et al., 1980). The Crank-Nicolson scheme adopted is simple and therefore was preferred to a slightly more accurate scheme.

Considering the equation of motion (Eq. 2) in total, it describes a so-called dynamic wave, in which inertia effects play an important role. It is essential to consider this aspect for rapidly changing conditions, such as tidal waves. The diffusion wave is commonly used for slowly changing flood waves. This means neglecting the first two terms of Equation (2), i.e. the inertia effect and the non-uniform flow. Physical properties of a flood wave can be used to determine the required wave type (GRIJSEN AND VREUGDENHIL, 1976). In the SIMWAT model the user can select either the dynamic wave or the diffusion wave type. The diffusion wave type is sufficient for hydrological modelling and has fewer restrictions on the time step to be used. Using Equation (2) and the Manning formula for the boundary friction (Eq. 5), the diffusion wave equation can be written as:

$$Q_{ji} = \frac{R^{4/3} A^2}{n^2 L_{ij} \, | Q_{ji}^* |} (h_j - h_i) \qquad (56)$$

where:

R = hydraulic radius (m)
A = wetted area (m^2)
h_i = water level in node i (m)
h_j = water level in adjacent node j (m)
n = coefficient of roughness ($s{\cdot}m^{-1/3}$)
L_{ij} = length of section between nodes i and j (m)
Q_{ji}^* = flow rate calculated in previous iteration ($m^3{\cdot}s^{-1}$)

In order to obtain a linear relationship, the flow rate from a previous iteration, Q_{ji}^*, is used as a known value in Equation (56). This means an iterative procedure must be used. Substituting Equation (56) into Equation (55) it can be written as:

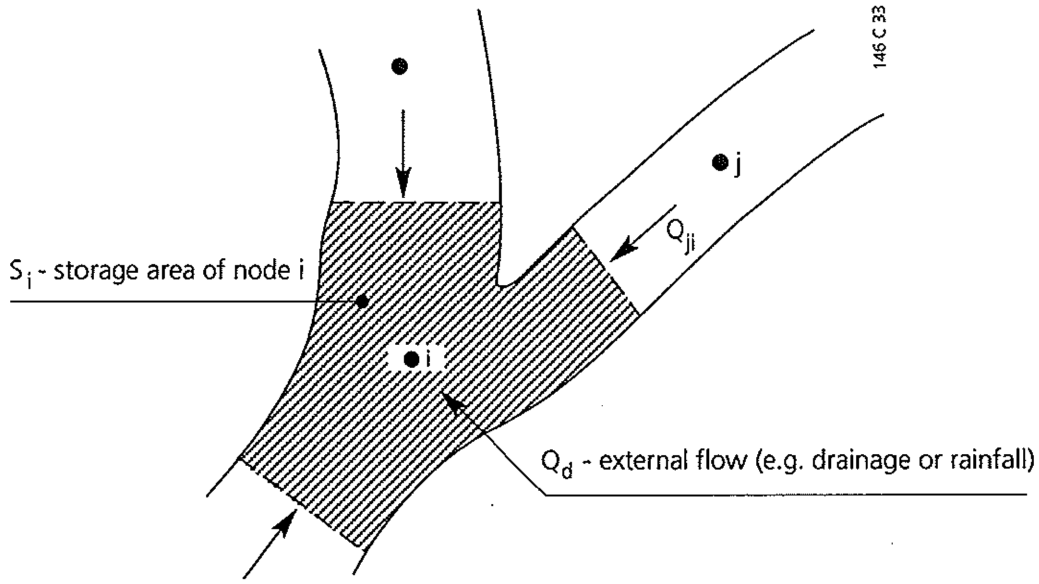


Fig. 33 *Water balance in junction of water courses represented by a nodal point. The incoming flow rates Q_{ji} and the external flow Q_d results in a change of water level over the storage area S_i.*

$$S_i \frac{\Delta h}{\Delta t_s} = (1 - W)Q_o^t + W\sum_j \frac{R^{4/3}A^2}{n^2 L_{ij}\,|Q_{ji}^*|}(h_j - h_i)^{t+\Delta t_s} + Q_d^t \tag{57}$$

where:
Q_o = flow rates from nodes j to node i at time t ($m^3 \cdot s^{-1}$)

Equation (57) can be re-arranged to get a set of simultaneous equations (N equations with N unknown water levels). The equation can be written for all nodal points in matrix form as:

$$\{T\} = [K_r]\{h\} \tag{58}$$

where:
$\{T\}$ = vector of all the known terms
$[K_r]$ = resistance and storage matrix
$\{h\}$ = vector with unknown water levels (m)

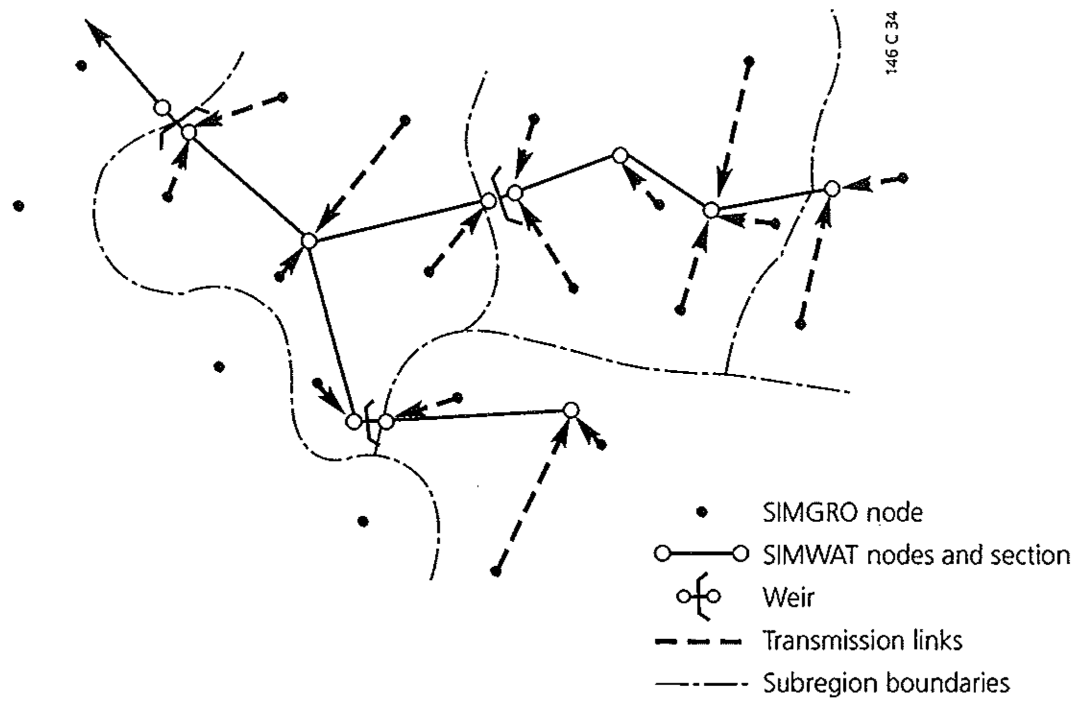
Matrix $[K_r]$ contains all contributions to the flow resistance between point i and its adjacent nodes and the storage in node i. Because the resistance factor in the matrix $[K_r]$ is a function of the water level, the solution is obtained by successive approximations. In general only a few iterations are required per time step. Equation (58) is solved by the Crout reduction method. To include special sections, such as weirs, pumps and gates, it is necessary to write the specific relations for such sections in the same form as Equation (58). Also in this way a prescribed water level or inflow can be kept for special nodes (QUERNER, 1986b).

4.5 Integrating the groundwater and surface water modules in the MOGROW model

The flow of water in open channels can change in a short time. It is a quickly responding system with rapid fluctuations in water level. This means that surface water models in general have a small time step, in the order of several minutes to one hour. Unlike flow in open channels, groundwater flow (phreatic) can change relatively more slowly. Taking a net precipitation infiltrating into the ground, most of this water is stored in the unsaturated and saturated zones. Therefore a hydrological model has a groundwater system that reacts slowly to changes, plus a surface water system with a quick response. Regional groundwater models of the saturated zone use time steps of one day to as much as ten days. Unsaturated flow, especially when we look at abrupt changes in time (heavy rainfall), would also require a much smaller time step. The pseudo-steady state concept used to model unsaturated flow is limited to minimum time steps of approximately one day. The saturated groundwater flow is, in principle, not limited in the size of the time step. But when modelling the interaction with the surface water the time step should be appropriate given the time step admissible for the surface water. Thus the two possibilities when using an integrated groundwater and surface water model are:
- use the smallest time step required by the groundwater or surface water systems,
- accept different time steps for both systems.

If one time step is used for both surface and groundwater it would have to be in the order of ten minutes to about one hour. This would mean an unacceptable computation time. The concept for water flow in the unsaturated zone, at present the pseudo-steady state approach, would have to be changed to a finite-difference approach. But this would make it impossible to achieve a relatively simple hydrological model; therefore the surface water system was given its own time step. The result is that the surface water module performs several time steps between two time steps of the groundwater module. The groundwater level is assumed to remain constant during that time and the interaction between groundwater and surface water is accumulated using the updated surface water level. The next time the groundwater module is called up the accumulated drainage or subsurface irrigation is used to calculate a new groundwater level.

The geographical link between both modules is shown in Figure 34a. Nodal points of the groundwater module are assigned to a nodal point of the surface water moda



b

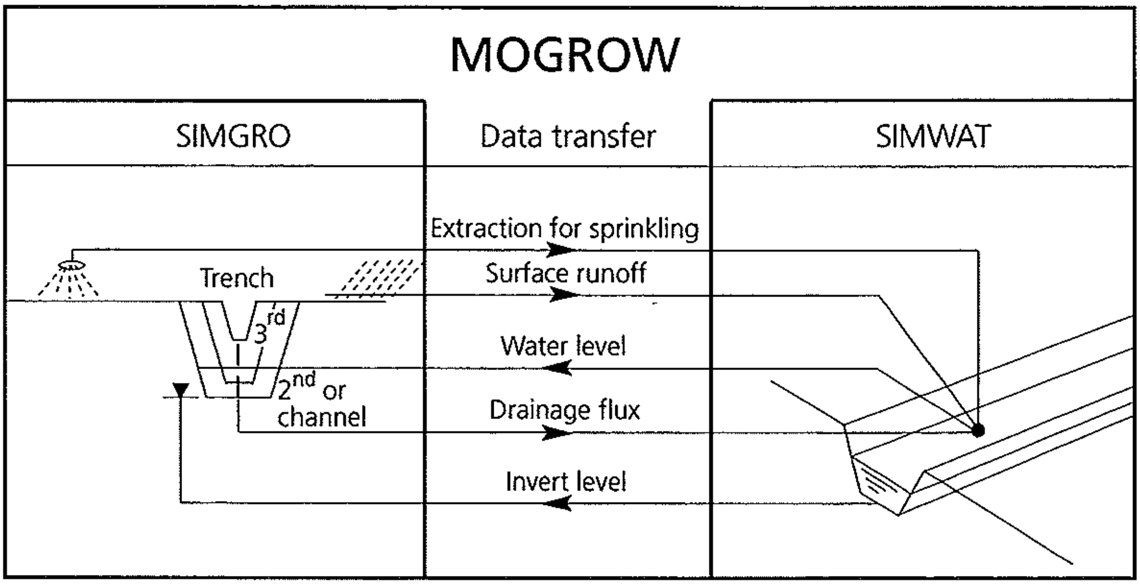


Fig. 34 *Geometrical links in the MOGROW model between the groundwater module SIMGRO and the surface water module SIMWAT necessary for the transfer of data.*
a) Transmission links between the finite element network of the groundwater system and the nodes of the surface water network
b) Transfer of input data (invert level of the channel or secondary system) and results between groundwater and surface water module

ule (transmission links). These links should consider sub-catchments or variation in surface water levels by means of weirs. The data initially transferred are the invert levels (bed level) from the surface water network that are to be used for the invert levels of secondary or channel system in the groundwater module (Fig. 34b). Time-dependent data (fluxes and water levels) are transferred between the two systems. The transfer is done in such a way that it does not require an inordinate amount of computer time. The surface water level calculated for a node is transferred to the connected nodes of the groundwater system. The drainage system (trenches, third, second and the channel system) uses this surface water level to calculate the drainage or subsurface irrigation. Subsurface irrigation from these subsystems is only possible when there is water in that system. Requirements for sprinkling from surface water and possible surface runoff are transferred to the surface water module each time step of the groundwater module (Fig. 34b).

5 Verification of the model MOGROW in the Hupselse Beek area

The modelling approach discussed in Chapter 4 had to be verified. Also its possibilities and the drawbacks needed to be quantified. Therefore I focused primarily on a model analysis and no calibration was carried out. The input data obtained from field measurements or the literature were used. Several aspects related to the flow behaviour in the unsaturated zone (theory described in Section 4.3.4), were tested. The model was applied to the Hupselse Beek area, using meteorological data for the year 1981. It was then validated, using data from the 1982-85 period.

5.1 Description of the catchment area

The Hupselse Beek catchment is in the east of the Netherlands near the border with Germany (Fig. 35). The Hupselse Beek is the upper reach of the river Berkel. The area covers 6.5 km^2 and lies between 24 and 33 metres above NAP (reference level in the Netherlands). The average slope of the area is about 0.8%. The river flows through a wide valley and with a relatively steep gradient of 0.06% to 0.25%. Land use is predominantly agricultural; about 70% is pasture, 21% is arable land (mainly maize) and 6% woodland. Within the catchment the main stream is 4 km long and has seven small tributaries varying in length between 300 and 1500 m (Fig. 35). The total length of all the water courses and ditches in the catchment is approximately 43 km and they cover about 1.7% of the total area.

The Hupselse Beek catchment was affected by glaciers which deposited boulder clay in an early Pleistocene period. Later, sands were deposited over the entire area, masking the relief of the underlying boulder clay surface. The most important deposits which can be distinguished are: aeolian (cover sand), glacigenous (till and fluvio-glacial), fluviatile (course sand and gravel) and marine (Miocene) deposits. The top of the impermeable layer (marine clays) is found at or near the surface in the eastern part of the area. Westwards the impermeable layer dips to a depth of about 8 metre.

In 1967 and 1968 the water management of the area was improved. Until then the area was poorly drained and flooding occurred frequently. After these improvements subsurface drains were also installed, especially in the eastern part. Several weirs were constructed to prevent erosion of the river bed. The specific design discharge was assumed to be 1.7 $l \cdot s^{-1} \cdot ha^{-1}$ (14.7 $mm \cdot d^{-1}$), having a frequency of excee-

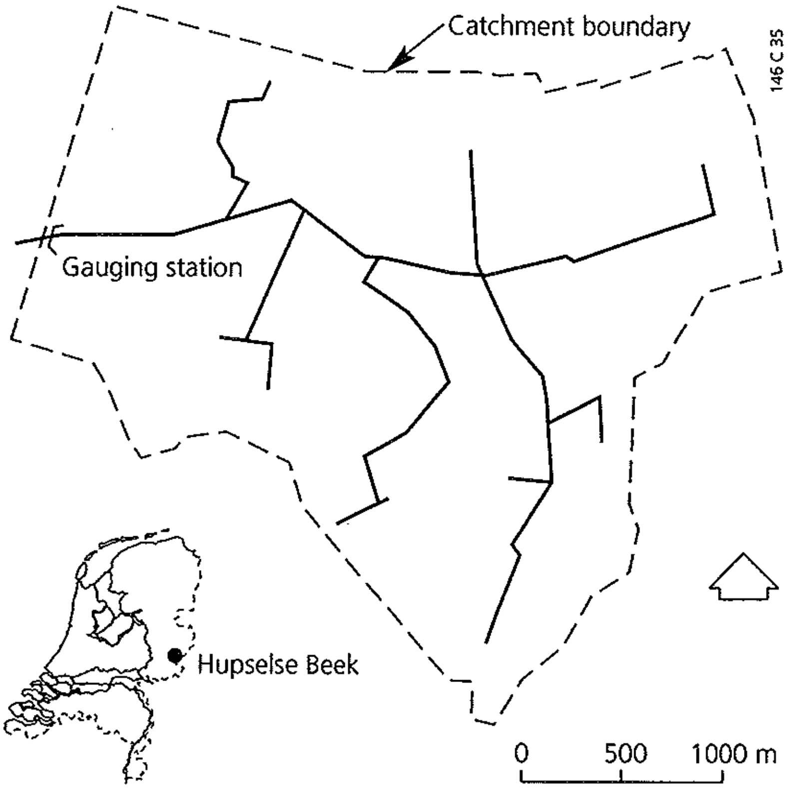


Fig. 35 The Hupselse Beek catchment and its major water courses.

dance of approximately once a year during 1 to 2 days (WERKGROEP AFVOERBEREKENINGEN, 1979). Of the two components of the hydrograph the immediate runoff is the most important and the delayed runoff is less important (Fig. 41).

Hydrological data have been collected over the last 20 years in order to carry out detailed hydrological studies (WARMERDAM et al., 1982). Numerous data are recorded, at intervals varying from 20 minutes to twice a month. Precipitation, temperature, sunshine duration, relative humidity, radiation, wind speed, groundwater levels and discharges are measured continuously. All these data are available from the Rijkswaterstaat (Institute for Inland Water Management and Waste Water Treatment). Both the potential and actual evapotranspiration have been computed from the raw data (STRICKER, 1981). The year 1981 had some heavy rainfall in mid summer, with dry weather before and afterwards. The summer of 1982 was dry, but 1985 was quite wet.

5.2 Input data

For the model MOGROW the groundwater system needs to be schematized by means of a finite element network. The network, comprising 283 nodes spaced about 200 m apart, is shown in Figure 36. For the surface water module only the major water courses present were considered, requiring 50 nodes, 5 typical cross-sections and 21 weirs. These water courses considered in the surface water module were used as the so-called secondary drainage system in the groundwater module. Data on the surface water system was obtained from the water board de Berkel (VAN MOURIK and DROOGERS, 1989).

Weed control is usually carried out by the water board three times a year. The first clearance is at the beginning of May, the second from mid June to mid July and the last clearance in October or November. The exact time of weed control depends on the actual weed growth, which reflects the weather conditions. For the Hupselse Beek the flow resistance k_M is assumed to vary between 34 $m^{1/3} \cdot s^{-1}$ in winter to 20

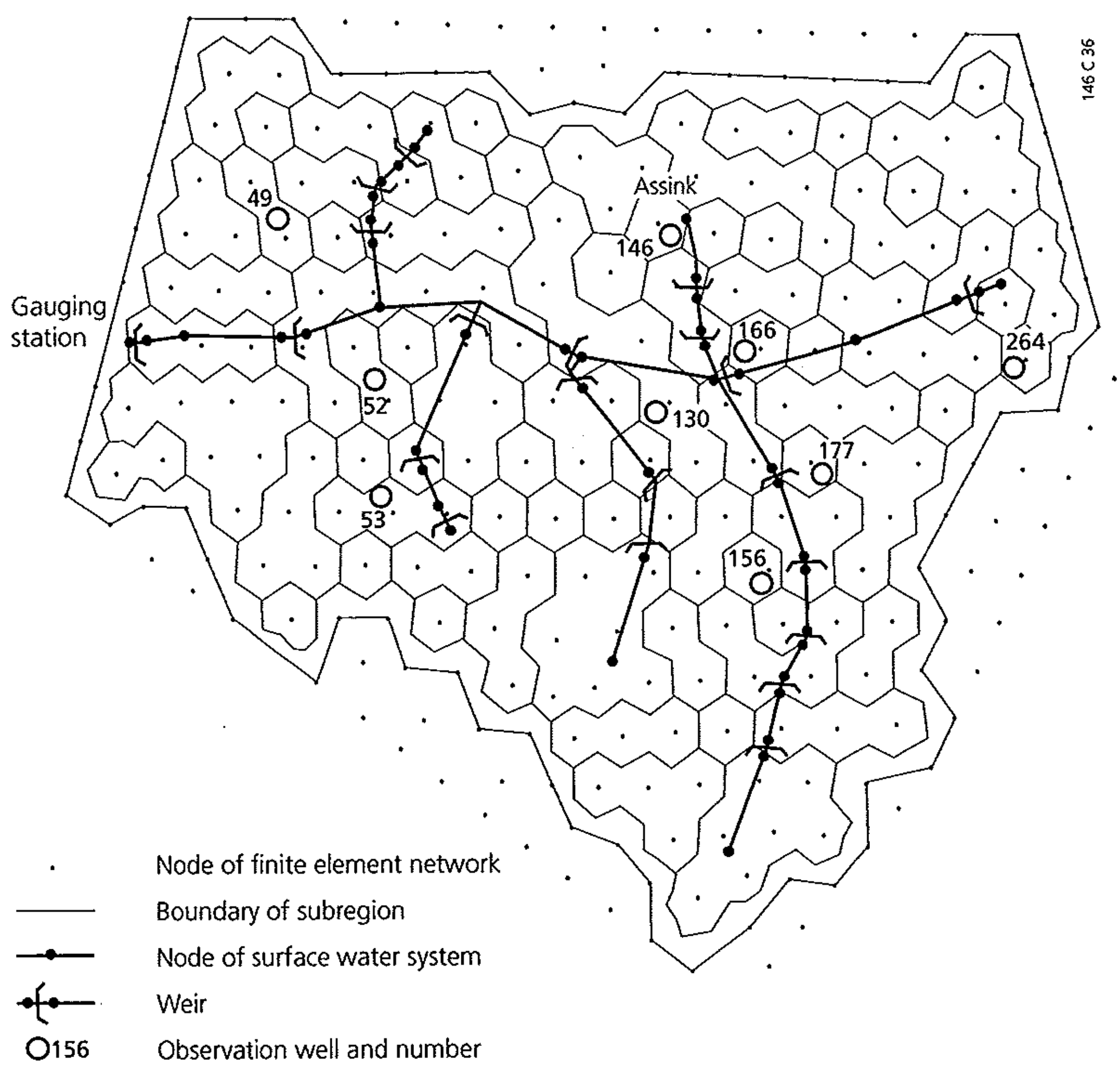


Fig. 36 *The Hupselse Beek catchment showing the schematization for the groundwater and surface water, together with the observation wells and gauging station. The subregions shown are for the schematization of the unsaturated zone.*

in summer (see Section 3.2). The tributaries were assigned k_M values of 30 and 14 for respectively the winter and summer periods.

The physical soil properties for the unsaturated zone module were obtained from WÖSTEN et al. (1983, 1985). From the mapping units 9 major pedological horizons could be distinguished, these were combined to 6 physical soil building blocks which were used to transform the existing soil map into a map of physical soil units (Table 3). The thickness of the sandy subsurface horizon varies with the presence of boulder clay. For each physical soil unit the actual characteristics of the unsaturated zone, i.e. upward flux, storage coefficient and equilibrium moisture storage of the root zone, were calculated using the CAPSEV model (Section 4.3.4). The relations obtained for soil unit 3 are shown in Figure 37. The equilibrium moisture storage varies slowly with the groundwater level below soil surface (Fig. 37a). The highly nonlinear relation for the upward flux can be seen. A typical relation for the storage coefficient is shown in Figure 37b. This storage coefficient applies to the unsaturated zone below the root zone, as shown in Figure 25. The dashed line in Figure 37b is the assumed transition from unity when the groundwater level is above soil surface, to very small when the groundwater level is just below the root zone.

The catchment was subdivided into 40 subregions using the physical soil properties and the layout of subcatchments (level for modelling the unsaturated zone). A further refinement, taking into account the variation in the depth of the groundwater level should not exceed 0.40 m, gave a total of 61 subregions (Fig. 36). This criterion was used, because the soil moisture flow in the unsaturated zone depends on the average depth of the groundwater (Section 4.3.4). Therefore this depth should not vary too much within a subregion.

Table 3 *Thickness of the building blocks (m) for each physical soil unit distinguished in the Hupselse Beek area (VAN MOURIK and DROOGERS, 1989; WÖSTEN et al., 1983).*

Description of building block (horizon)	Physical soil units					
	Plaggept		Haplaquod			
	1	2	3	4	5	6
Sandy surface	0.40	0.40	0.25	0.25	0.25	0.20
Sandy subsurface	2.10	0.80	2.25	1.35	0.95	0.60
Clayey	-	1.30	-	0.90	1.30	2.10

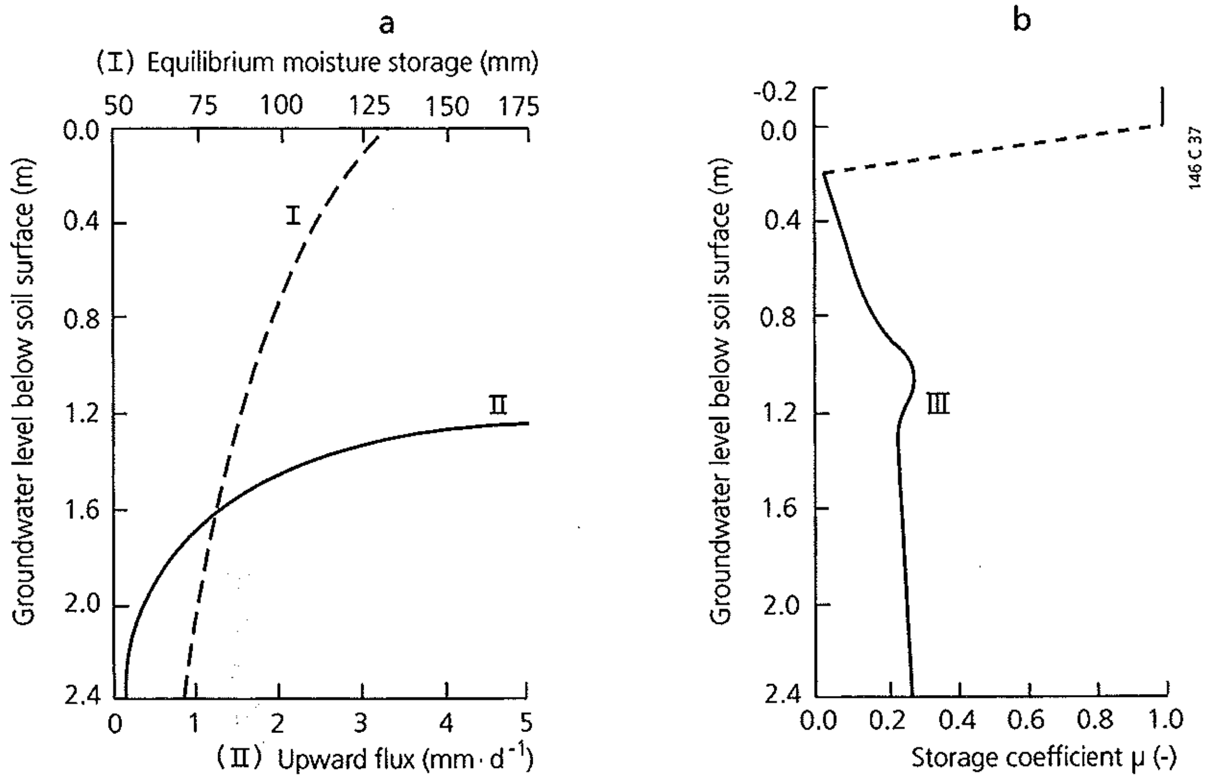


Fig. 37 *Characteristics of the unsaturated zone, which depend on the depth of the groundwater level (physical soil unit 3 from Table 3 and root zone thickness of 0.25 m).*
a) Equilibrium moisture storage of root zone (I) and upward flux from phreatic level to reach underside of root zone (II). The upward flux is limited to 5 mm·d^{-1}
b) Storage coefficient for subsoil (III). For dashed line see text.

The unsaturated zone was considered for the five land uses: pasture, maize, cereals, sugar beet and woodland. The thickness of the root zone for the different types of land use and the physical soil units is given in Table 4.

The transmissivity (T) measured at a number of locations within the catchment has been found to vary between 10 m^2·d^{-1} in the east to 350 m^2·d^{-1} in the north-west (KRUITWAGEN and SWINKELS, 1986). Given this thickness (STUIP and BOEKELMAN, 1976) the saturated hydraulic conductivity is in the order of 5 to 40 m·d^{-1}. Initially the conductivity was assumed to be 10 m·d^{-1} for the entire region.

The drainage resistance can be calculated from hydrological parameters and the ditch spacing (Section 4.2.2). The smaller ditches (all tributaries to the main stream) can be considered to have an average spacing of about 185 m. For this ditch spacing, the sum of horizontal and radial resistances (Eqs. 34 and 35), is in the order of 70 to 120 days.

The drainage characteristics of the catchment can also be found by plotting the depth of the groundwater against the discharge. The result, being an average depth

Table 4 *Thickness of root zone (m) for the different types of land use considered in the model MOGROW (VAN MOURIK and DROOGERS, 1989).*

Land use	Thickness of root zone for physical soil units		
	1 and 2	3, 4 and 5	6
pasture	0.30	0.25	0.25
maize and cereals	0.40	0.25	0.25
sugar beet	0.35	0.25	0.25
woodland	1.00	1.00	0.50

from 9 observation points for the period 1981 to 1985, is shown in Figure 38. Line I in Figure 38 can be regarded as the regional flow component (base flow), which is the drainage from the main stream. Line II represents the flow to the main stream and its tributaries (secondary system) and line III to the smaller ditches and to the subsurface drainage system (tertiary system). Points laying below line III indicate discharges other than from drainage; such as surface runoff. Another cause may be a variation in specific discharge within the catchment. The average drainage resistance derived from Figure 38 is approximately 450-600 days for the secondary system (difference between line I and II) and 70-90 days for the tertiary system (difference between line II and III). The intersection of line II and III (Fig. 38) can be

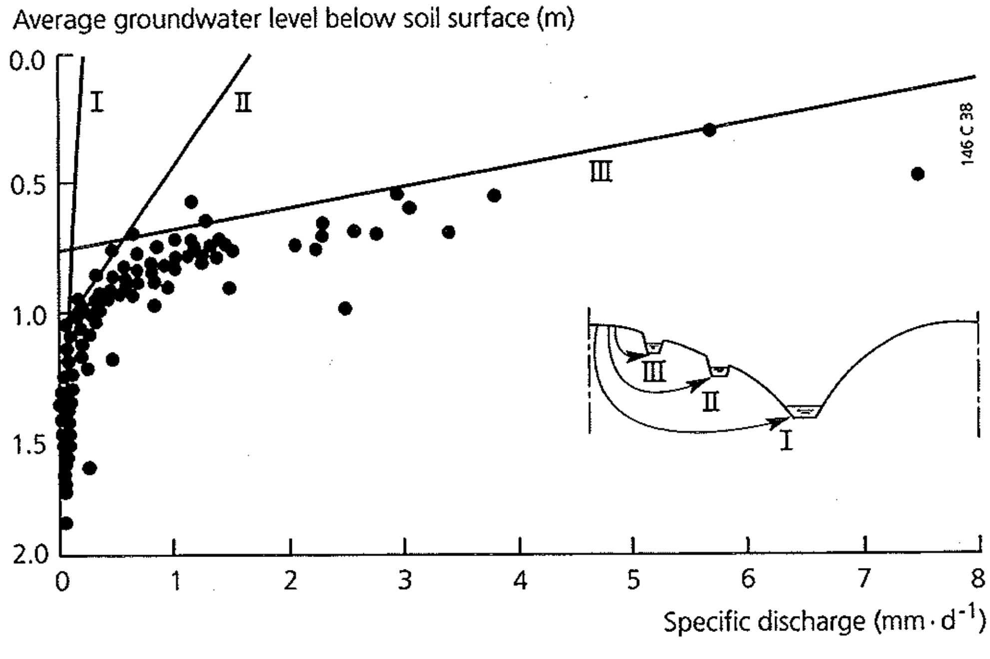


Fig. 38 *Relation between specific discharge and the average depth of the groundwater level to get an insight into the drainage characteristics to three sub-systems. Data from 9 observation wells in Hupsel catchment from 1981-1986.*

regarded as the bottom of the tertiary system (including subsurface drains) and the intersection of line I and II as the bottom of the secondary system. This would give a depth of 0.55-0.70 m for the tertiary system and 0.90-1.05 m for the secondary system. Some field measurements in the catchment yielded depths of 0.65 to 0.85 m for the tertiary system and depths of 0.9-1.20 m for the secondary system. The procedure described here, to estimate drainage resistances, is only allowed when the catchment is homogeneous.

The input data required for the MOGROW model are summarized in Table 5. The simulation with this data set is called the reference simulation. All these data are used in Section 5.3 to compare the model results with observed data.

Table 5 *Overview of data input to the MOGROW model and used for the reference simulation. The source of data is given by a code M (from maps), F (field data) and L (literature). The data required in the schematization are given the code N (nodes of finite elements), S (subregions), U (land use and soil unit), W (water course) and C (typical cross-section of water course).*

Parameter	Source of data	Required in schematization	Value
Surface water module (major stream)			
Invert levels	M*	W	
Dimensions water courses and weirs	M*	W	
Flow resistance	L	C	14 - 34 $m^{1/3}.s^{-1}$
Groundwater module			
Land use	F**	S	
Physical soil properties	F	S	Table 3
Thickness of root zone	L	U	Table 4
Ground level	M	N	
Sat. hydr. conductivity	F	N	10 $m \cdot d^{-1}$
Thickness aquifer	F	N	2 - 12 m
Drainage resistance secondary system	F	N	500 d
Depth of tertiary system	F	N	0.65 m
Drainage resistance tertiary system	F	N	75 d
Area with subsurface drains	M**	N	
Meteorological data, groundwater levels and discharges	F**		

* Water board de Berkel
** Rijkswaterstaat

5.3 Comparison of calculated and observed variables

Observations of groundwater levels and discharges were used to verify the MOGROW model. The moisture storage in the root zone and the reduction in evapotranspiration (both measured and calculated) were compared as well. The computations were done with daily meteorological data for 1981. The time step for the groundwater module was one day and for the surface water module half an hour. It took approximately 30 minutes of CPU time to simulate one year on a VAX 4200.

Groundwater levels

The mean standard deviation (root mean square) was used as a measure of the agreement between the observed and calculated levels. A problem faced by the comparison is the fact that the observed groundwater level is for a certain location, whereas the calculated level is an average for the area associated with a nodal point (Fig. 39). In order to be able to compare point and average values a conversion factor could be introduced. This factor would have to be time dependent, as it is related to the difference in head between the surface water and the groundwater level midway between two ditches. The position of the observation point in relation to the surface water system is also important when estimating this conversion factor (Fig. 39). And the variation in ground level between nodal points in relation to the location of the observation point may influence this conversion factor too. I decided not to use a conversion factor, because of these uncertainties.

The difference between observed and calculated groundwater levels was analysed for nine observation wells shown on Figure 36. The groundwater hydrographs for two locations are shown in Figure 40. Nodes 53 use observations taken once in 14 days, but daily observations are used for node 146 (Assink meteorological site). The differences between calculated and measured groundwater levels are quite small,

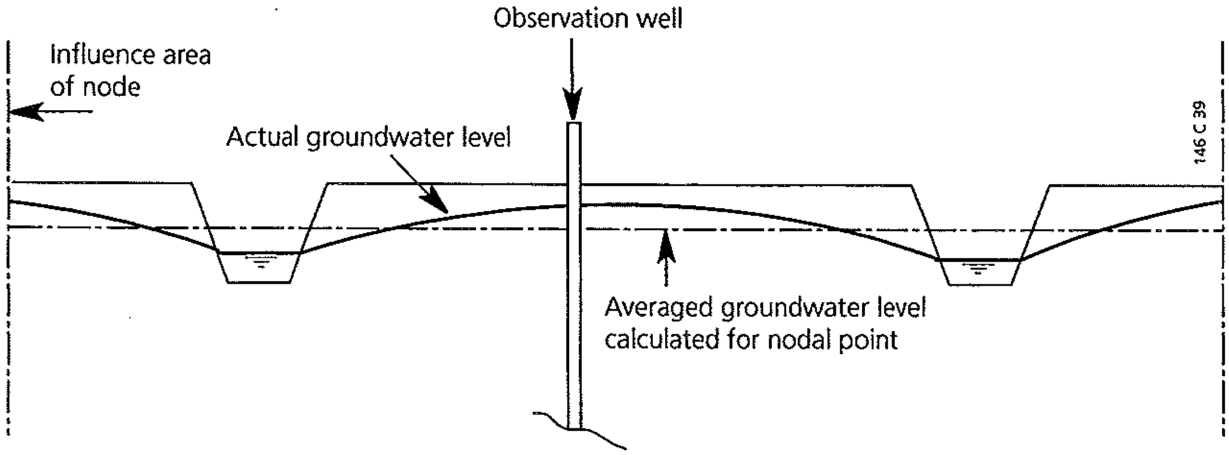


Fig. 39 The actual groundwater level (solid line) shown differs from the average level calculated for a nodal point (dashed line).

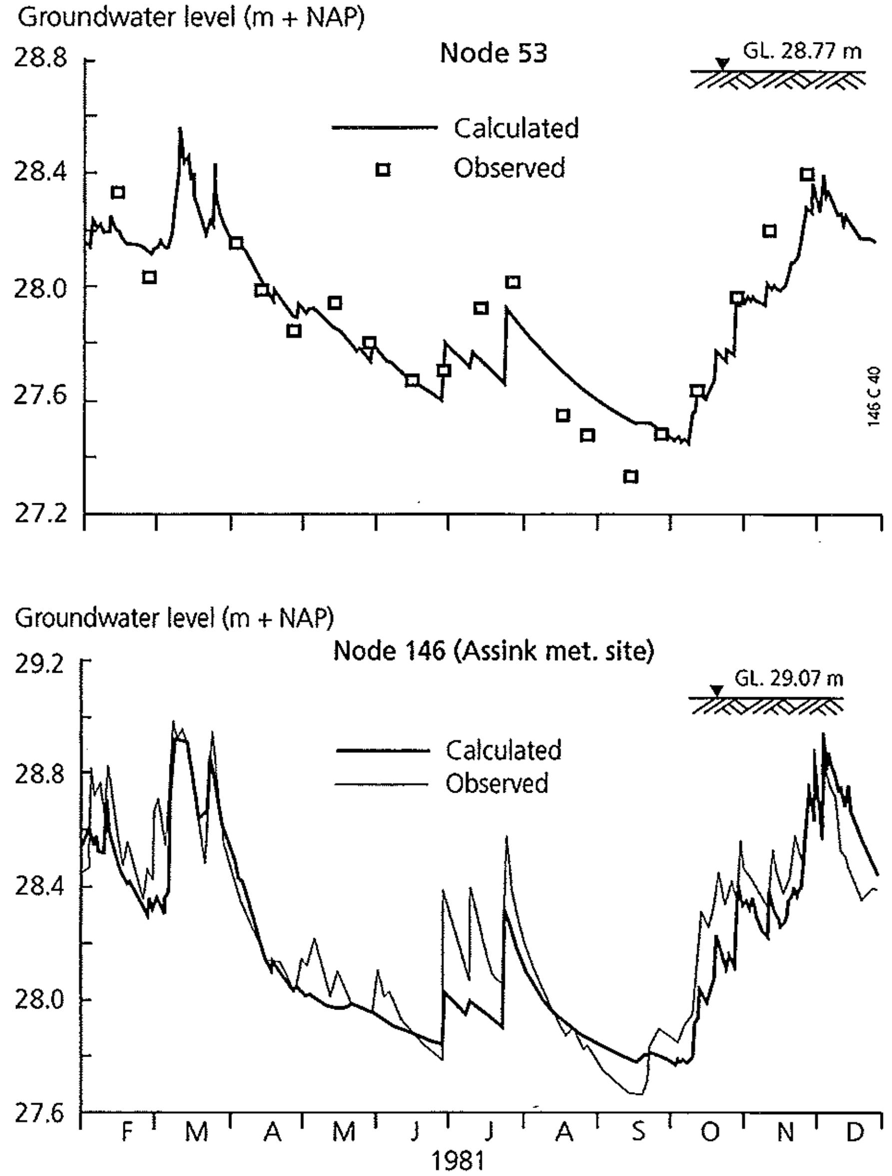


Fig. 40 *Calculated and observed water table for 1981, using the data set from Table 5, for nodes 53 and 146. Observation wells are shown in Figure 36. Daily observed groundwater levels were used for node 146 (Assink met. site).*

but increase after short periods of heavy rainfall (summer 1981 - Fig. 40). The peaks calculated are lower than observed, partly because the comparison is between point values (observed) and average values (calculated). After a dry period the calculated levels are lower. The differences in terms of standard deviation are given in Table 6a. The calculated and observed groundwater levels compare well, but two points, nodes 177 and 49, have a standard deviation greater than 0.30 m (Table 6a). These high values suggest that either the input data are wrong or the schematization is inadequate in the vicinity of these two points. Six points have a deviation of 0.20 m and less. Deviations of less than 0.20 m are regarded from

Table 6 *Mean standard deviation (root mean square) for the reference simulation, together with minimum, maximum and average differences in (a) groundwater levels and (b) discharges for the met. year 1981 (input data given in Table 5; location of nodal points see Fig. 36).*

a) Groundwater levels

Nodal point	Mean deviation (cm)	Observed minus calc. Min (cm)	Max (cm)	Average (cm)
49	31	-19	58	16
146	15	-26	21	-3
166	25	-21	52	10
264	20	-16	35	12
177	41	-72	-20	-38
156	19	-23	41	11
130	12	-20	31	-2
52	19	-41	10	-13
53	11	-19	21	1
all	22	-28	32	12

b) Discharges

Amount of discharge ($mm \cdot d^{-1}$)	Mean deviation ($mm \cdot d^{-1}$)	Observed minus calc. discharges (average) ($mm \cdot d^{-1}$)
> 2.0*	1.98	0.04
< 2.0	0.30	

* 29 days discharge above 2 $mm \cdot d^{-1}$

experience as good agreement because in those cases the calculated groundwater levels show the same reaction in time as observed. Also given in Table 6a are the minimum and maximum differences that occurred within the calculation period, and the average difference. For seven observation points the average difference is less than 0.15 m.

Discharge

The hydrograph observed and calculated is shown in Figure 41. For the analysis of the outflow hydrograph a differentiation was made into mean daily peak flows (> 2 $mm \cdot d^{-1}$) and the low or base flow (< 2 $mm \cdot d^{-1}$). Comparison of the peak flows gave a deviation of 1.98 $mm \cdot d^{-1}$ occurring in 29 days in 1981 (Table 6b), which is an appreciable disparity. The duration of one flood is also very short; only a couple of days at the most. Because the calculations are based on a time step of one day for the saturated and unsaturated zones, this difference is reasonable. It is more important to compare the difference in the peaks. This aspect is discussed further as part of the model analysis (Section 5.4). The low flow conditions (< 2 $mm \cdot d^{-1}$) compare much better. The deviation of 0.3 $mm \cdot d^{-1}$ can be regarded as sufficiently small. The base flow condition as shown in Figure 41 (occurring June, August and September 1981) has an average difference of 0.08 $mm \cdot d^{-1}$, which is reasonable. Taking the major flow components from the catchment for a small period in mid August, gives the results shown in Figure 42. Ninety per cent of the groundwater goes to the

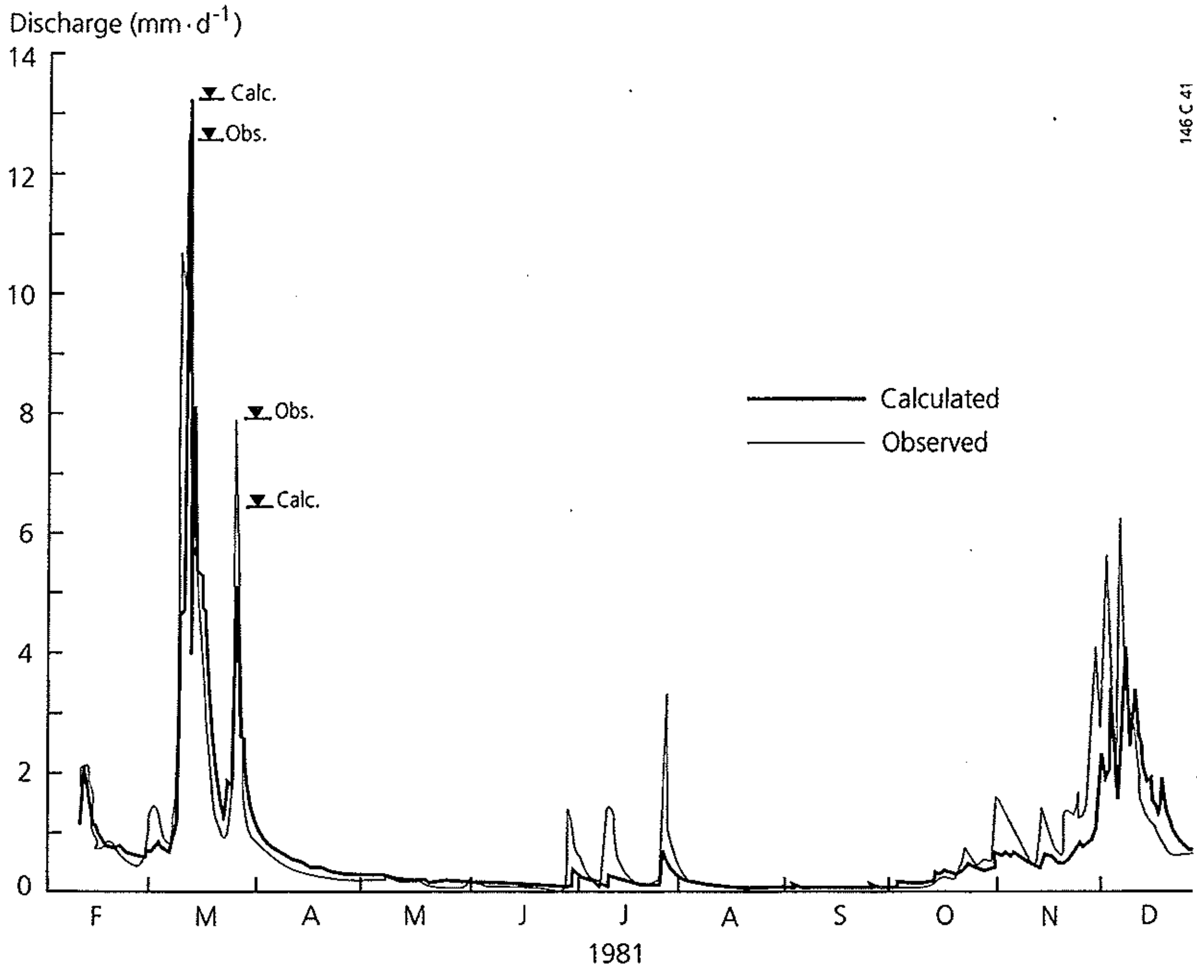


Fig. 41 *Calculated hydrograph of reference simulation (data set in Table 5), together with the observed hydrograph for the Hupsel catchment (met. year 1981).*

unsaturated zone as an upward flux and only a small portion is base flow.

Moisture in root zone

The variation in calculated moisture storage over two years was compared with measured values. Neutron probe data were available for depths of 0.10; 0.20 and 0.30 m from the Rijkswaterstaat. Measurements at 0.10 m depth are greatly influenced by disturbance to the soil (e.g. inhomogeneity) and to the soil surface. When estimating the total moisture storage for the root zone up to a depth of 0.30 m, the values for these three depths were therefore given weighting factors of respectively 0.2, 0.6 and 0.2. The calculated and measured moisture storages for node 146 (Assink met. site) and node 130 (Ten Barge site) are shown in Figure 43. In general a good correlation was found. Certain measurements show moisture storages higher than the equilibrium storage used in the model (some points marked on Fig. 43a). These points were all measured after days with heavy rainfall. A rather low value was also measured within a frosty period. If these values are excluded the agreement is good, except for the summer period of 1982. In a dry period the moisture

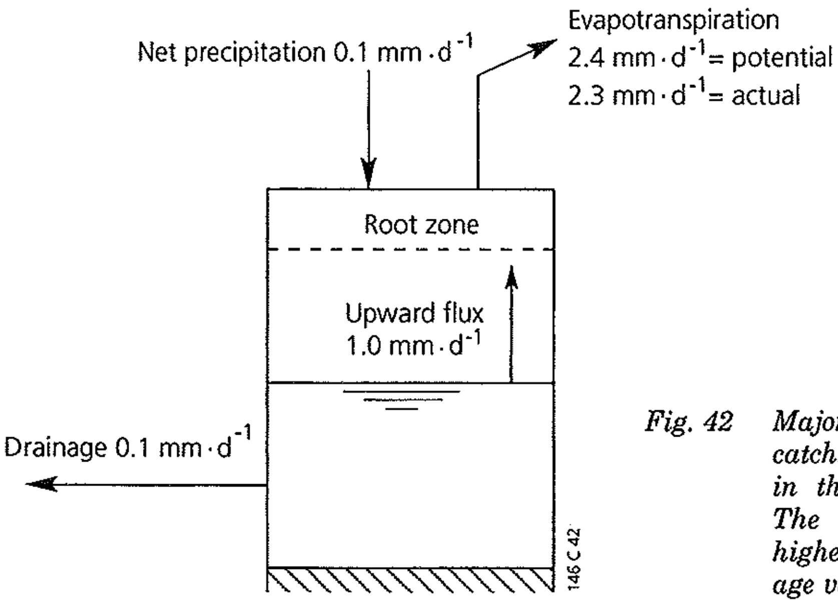


Fig. 42 *Major flow components of Hupsel catchment for a ten-day period in the middle of August 1981. The upward flux is ten times higher than the drainage (average values in mm·d^{-1}).*

storage in the root zone depends on the upward flux, the depth of the groundwater level below soil surface and the storage characteristics of the soil. The standard deviations between calculated and observed moisture storages are given in Table 7. At the Assink site soil unit 3 (Table 3) is present. Soil units 4 to 6 were also used for this location, to obtain the possible variation in moisture storage for these soils. For the Assink site the lowest standard deviation was obtained for soil unit 5. The soil schematization becomes important when the clay layer is close to the surface (soil unit 6). The upward flux is then reduced, resulting in a low moisture storage in dry periods and therefore a large deviation (Table 7).

Evapotranspiration

Data were available on evapotranspiration calculated as potential and actual for grass (node 146 - Assink met. site; KOOPMANS et al., 1990). No differences

Table 7 *Mean standard deviation (root mean square) of moisture storage in the root zone, together with the average difference between calculated and observed moisture storage, for a root zone of 0.30 m thick (met. years 1981 and 1982; locations shown in Fig. 36).*

Location	Soil unit	Mean deviation (mm)	Observed minus calculated average (mm)
Assink	3	13	-4
	4	13	-4
	5	11	-1
	6	23	19
Ten Barge	2	11	-7

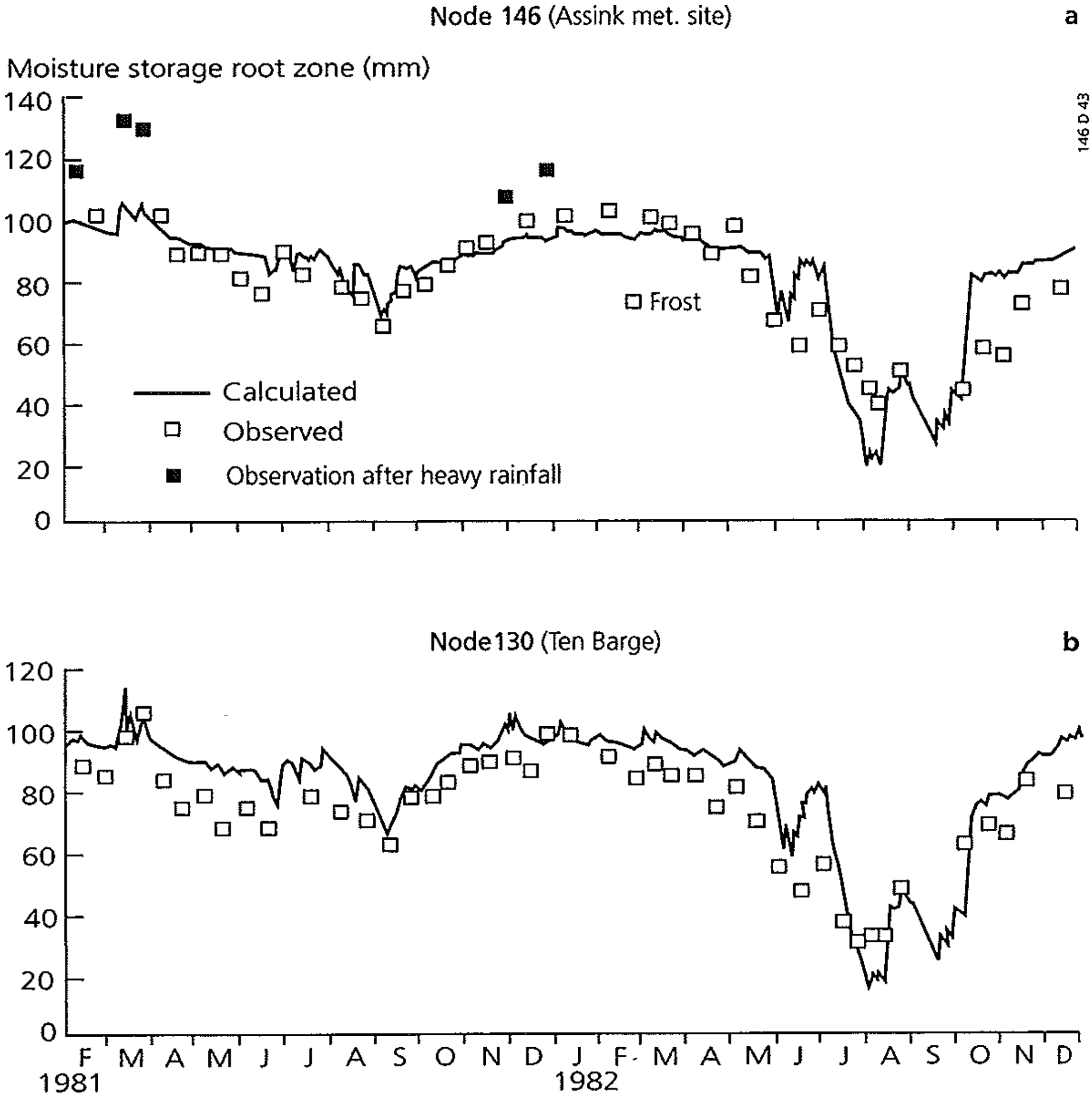


Fig. 43 Comparison of measured and calculated moisture storage for the root zone below a grass cover (thickness root zone 0.3 m; location of nodes shown in Fig. 36).
a) Node 146 (Assink met. site) with physical soil unit 3
b) Node 130 (Ten Barge site) with physical soil unit 2

between potential and actual evapotranspiration occurred in 1981, so did the model MOGROW. The reduction for 1982 is appreciable (Table 8). The overall difference between model results and field data is small. The reduction in evapotranspiration given by the model for September is too large and was caused by the lower moisture storage calculated for the root zone (Figure 43). The upward flux might be somewhat small for the deeper groundwater levels. And the calculated groundwater level might also be slightly too low than in reality, which in turn gives a smaller upward flux. These aspects could be caused by the schematization of the unsaturated zone into root zone and subsoil, or by the physical soil properties.

Table 8 *Reduction in evapotranspiration (mm) derived from field data and calculated with the MOGROW model (summer period of 1982 for node 146, which is Assink met. site).*

Month	Potential evapotr.*	Reduction in evapotranspiration	
		From field data*	MOGROW model
July	99	7	7
August	81	23	20
September	55	3	7
October	25	-	1
Total	260	33	35

* After KOOPMANS et al. (1990)

5.4 Model analysis

A combined surface and groundwater flow model of this size and nature has many input parameters at so many locations (each nodal point and subregion). The output of such a system is affected inherently by uncertainty due to:
- model simplification and schematization;
- inaccuracy of the input data.

Users of a hydrological model like to have some idea about the spread of output values and about how sensitive the outputs are to the uncertainty of input data. Parameter estimation can be used, but this is problematical for regional hydrological models because:
- it is difficult to specify appropriate criteria for parameter selection;
- the parameters are interrelated;
- many computations are required;
- the limited amount of field data;
- the pronounced seasonality in hydrological regimes.

The minimization of a sum of squares of deviation as the objective function is a parameter estimation technique commonly used in hydrology. Search techniques are then employed to find the parameter set that optimizes this objective function. It is also difficult to find a parameter set that meets multiple objectives, i.e. fits peak flows, base flows, groundwater levels and evapotranspiration in an 'optimal' manner. The use of multiple objective criteria for parameter estimation allows more of the information contained in the data set to be used. It was beyond the scope of the research reported here to go into these aspects deeply. Therefore instead I analysed the model by firstly comparing calculated and measured data in detail, in order to find the deficiencies in either the schematization or the processes considered. I then carried out a sensitivity analysis.

The calculations given in Section 5.3 show the differences between calculated and observed groundwater levels and discharges. The typical disparities noted are:
for the groundwater levels:
- in summer the observed levels are lower than calculated;
- in autumn the calculated levels recover more slowly than the observed levels;
- observed levels fluctuate more than calculated levels;

for the discharge:
- the peak flows calculated are slightly less than observed (winter);
- heavy rainfall in July (3 storms of respectively 42, 26 and 23 mm) yields calculated peak flows far smaller than observed;
- the base flow calculated is slightly larger than observed.

The calculated variation in groundwater level is less than observed. This is partly attributable to the comparison being between observed point values and calculated average values, as was shown in Figure 39. The other possible cause is that the storage capacity in the unsaturated zone is overestimated.

The calculated discharge shown in Figure 41 remained relatively low in July while the observed discharge had an appreciable peak. This might be the effect of surface runoff, interflow, hysteresis, preferential flow, or a combination of these. To analyse the observed deficiencies, calculations were done including each of these aspects singly. The underlying theory and the implementation in the hydrological model were described in Sections 4.2.2 and 4.3.4. The results of these experiments were compared with the reference set of results (Fig. 40 and Table 6). Table 9 gives the assumptions and significant results. The change in mean standard deviation, as compared with the reference run, is given in Table 10. From the results it can be concluded that the effects give minor changes in the groundwater levels, but the change in discharge is more pronounced. The deviation of peak flows (> 2 $mm \cdot d^{-1}$) becomes smaller, but for the preferential flow the reduction is appreciable (Table 10b). Surface runoff, being 2-3% of the precipitation, gives an improvement, without any effect on the actual peak flow on 11 March. Heavy rainfall in March results in a measured peak flow of 12.6 $mm \cdot d^{-1}$. Interflow (perched water tables) improves the groundwater levels in certain parts of the catchment (Table 10a). But the peak flow in March is considerably underestimated. This may be partly because interflow in the unsaturated zone is treated using the time step of one day. The additional reservoir introduced by this concept in combination with the time step of one day, delays the peak flow.

Table 9 *Evaluation of results obtained with the special processes included in the unsaturated zone module of MOGROW.*

Assumptions	Effect		Result*
<u>Surface runoff</u>			
2 % of rainfall (10-20 mm·d^{-1})	gr w :	-	.
3 % of rainfall (> 20 mm·d^{-1})	disch:	higher peak flows all seasons	+
<u>Interflow (perched water table)</u>			
vertical resist. = 100 d	gr w :	generally lower levels	—
horizontal resist. = 50 d	disch:	summer peaks higher	+
only physical soil units 5 and 6		winter peaks lower	—
top of resistant layer is top of clay layer (Table 3)			
<u>Hysteresis</u>			
for wetting 12% decrease in water storage at pressure head of -10^2 cm (Fig. 22)	gr w :	higher levels summer and autumn	+
Equivalent moisture storage for root zone (Fig. 44)	disch:	slight increase of flow in autumn	—
		higher peaks in summer	+
Increase in soil moisture storage after wetting is 0.25 mm·d^{-1}			
<u>Preferential flow (Fig. 29)</u>			
fraction bypass: 15% of net precipitation	gr w :	slightly lower levels	.
fraction to drains: 0-18% of percolation	disch:	higher peaks (direct response)	++

* + = a better fit with observed data
— = worsening of fit with the observed data
. = no significant change from the reference simulation

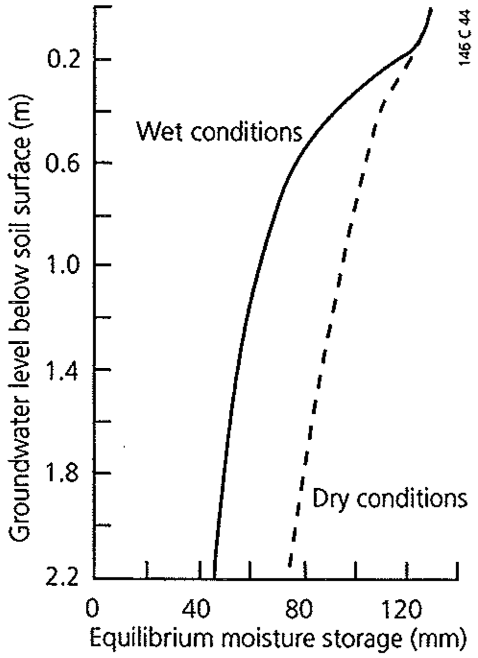


Fig. 44 *Equilibrium moisture storage of root zone for hysteresis effect (physical soil unit 3 from Table 3 and root zone thickness of 0.25 m).*

Table 10 *Change in standard deviation for (a) groundwater levels and (b) discharges when using special processes in the unsaturated zone. For reference simulation the standard mean deviation is taken from Table 6. Assumptions used for special processes are listed in Table 9 (met. year 1981; location of nodal points see Fig. 36).*

a) Groundwater levels (cm)

Nodal point	Reference simulation	Surface runoff	Interflow	Hysteresis	Preferential flow
49	31	2	-1	-2	1
146	15	0	0	2	1
166	25	2	8	-6	5
264	20	0	13	0	8
177	41	0	-19	2	-13
156	19	1	11	-4	5
130	12	1	1	1	9
52	19	-1	-2	0	-1
53	11	1	5	3	6
all	22	0	2	-1	3

b) Discharges ($mm \cdot d^{-1}$)

Amount of discharge	Reference simulation	Surface runoff	Interflow	Hysteresis	Preferential flow
> 2.0	1.98	-0.09	-0.21	-0.05	-0.21
< 2.0	0.30	-0.01	0.06	0.00	-0.07
peak*	12.0	0.1	-4.1	-0.3	-0.3
base**	0.08	-0.02	0.01	0.01	0.01

* observed peak flow for 11 March 1981 is 12.6 $mm \cdot d^{-1}$

** observed base flow for ten days in Aug 1981 is 0.05 $mm \cdot d^{-1}$

The effect of direct pathways (preferential flow) as well as interception by the subsurface drains could well explain the direct response of the Hupsel catchment after rainfall. This type of behaviour was treated in the model using the concept shown in Figure 29: rapid movement through the macropores and a minor part flowing through the soil matrix. The topsoil in the catchment has also been found to be strongly water-repellent (BRONSWIJK et al., 1990), which generally results in preferential flow. The simulated hydrograph, including preferential flow in the unsaturated zone, is shown in Figure 45. The differences in peak flow compared with the reference simulation as shown in Figure 41 are appreciable. In autumn the calculated discharges are slightly larger than observed. This overestimation is probably caused by the method used to calculate the preferential flow (as a direct

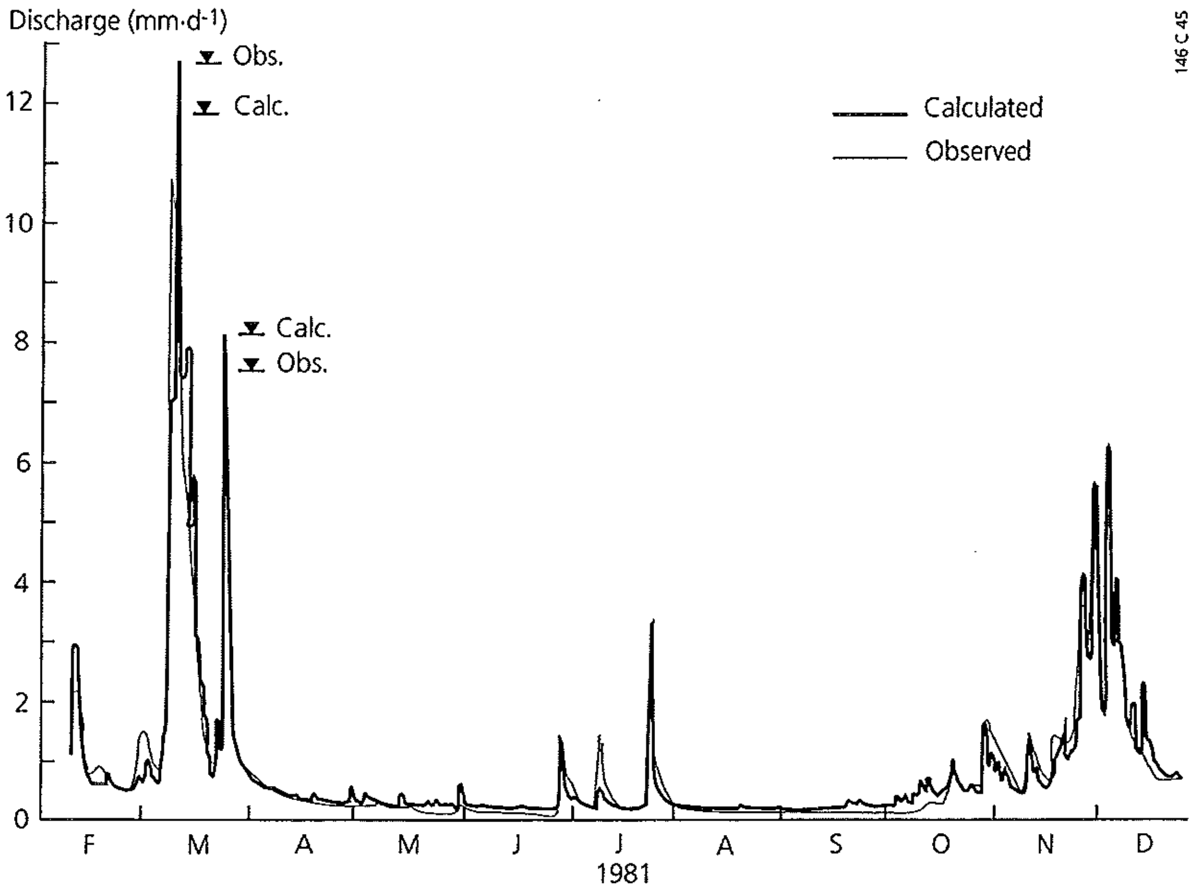


Fig. 45 Calculated hydrograph when using preferential flow process and interception by subsurface drains in the unsaturated zone, together with the observed hydrograph (met. year 1981).

link between precipitation and the groundwater system). After dry periods the retention of the preferential flow due to storage in the unsaturated zone will be significant.

Short duration rainfall

A rainfall event of short duration in May 1984 was evaluated with the MOGROW model in more detail. The calculation with the MOGROW model started from 1 January 1984, to ensure there was no influence from the conditions assumed at the start of the calculations (e.g. groundwater levels and moisture storage of root zone). The meteorological data used for the calculations were the hourly observations. The time step for the groundwater module was taken as 1 day and 3 hours. The surface water module used a time step of 15 minutes. At first the data set shown in Table 5 and a time step of 1 day for the groundwater were used. Some measured hydrological data together with the results calculated for a 10-day period in May 1984 are shown in Figure 46. The retention in the catchment is significant, from 70 mm of rainfall only 17 mm was observed as the discharge. The calculated drainage was 8 mm for the entire period (Fig. 46). The hydrograph for the period is shown in

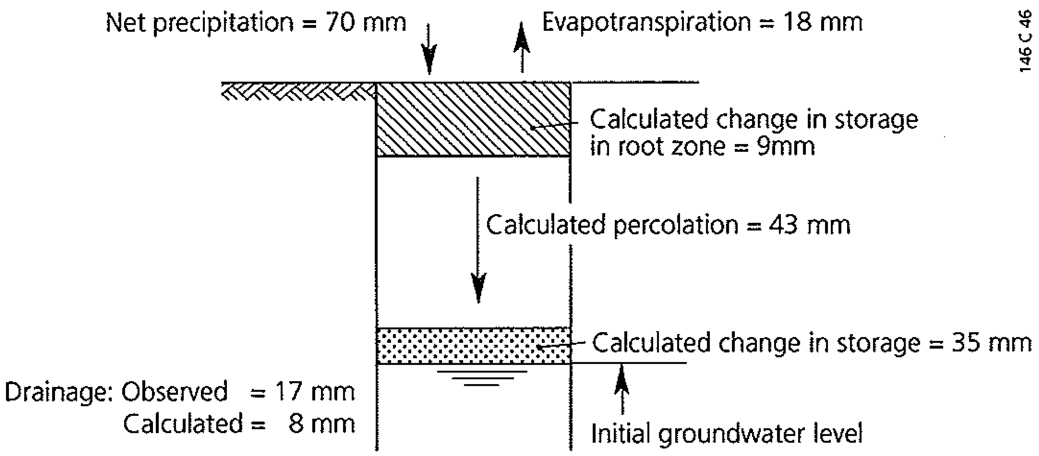


Fig. 46 *Observed and calculated fluxes for a 10-day rainfall period in May 1984, showing the retention characteristics of this period. The data set of Table 5 was used for the calculation.*

Figure 47. An additional run, using preferential flow and surface runoff (characteristics given in Table 9) is also shown. The simulation without preferential flow and surface runoff gives hardly any response for the discharge. Adding preferential flow and surface runoff gives an outflow hydrograph which is of the same order as the one obtained from observed data (Fig. 47). Using a time step of 1 day gives a peak flow smaller than observed. The 3-hour time step gives calculated and observed hydrographs of nearly the same shape (Fig. 48).

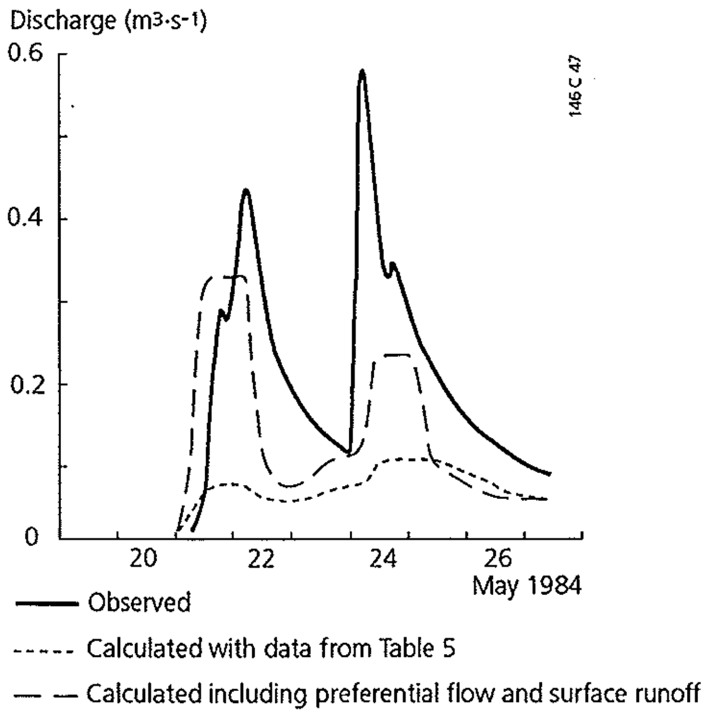


Fig. 47 *Observed and calculated hydrographs for rainfall event in May 1984, using the data set of Table 5 and including preferential flow and surface runoff. The time step for the groundwater module (Δt_g) was 1 day.*

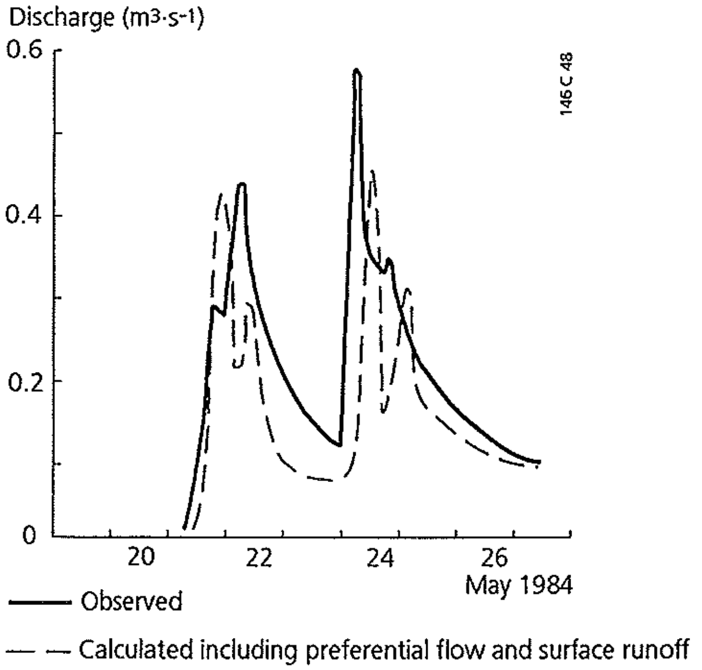


Fig. 48 *Observed and calculated hydrographs for rainfall event in May 1984, including preferential flow and surface runoff for a time step of 3 hours for the groundwater module (Δt_g).*

The success of modelling the actual hydrograph greatly depends on the conditions prior to the period being modelled. Groundwater levels and moisture storage in the root zone control the retention and thus the outflow. The total discharge calculated as shown in Figure 48 is slightly less than observed. This can be attributed to less land drainage from some parts of the catchment, as a result of the model having a larger storage capacity than in reality. These aspects only become apparent when discharges are available from subcatchments as well.

Sensitivity analysis

Before the model can be used with confidence in situations with less observed data, it has to be proved that the parameter values do not pose an undue problem. Specifically the uncertainty in the values need to be analysed. This qualitative assessment of the input parameters was carried out in the form of a sensitivity analysis. A single parameter was varied each time from its best known value used in the reference simulation, either measured or taken from the literature (Table 5). The effects on groundwater levels and the simulated hydrograph were then assessed by the mean standard deviation. The sensitivity analysis included the following parameters:

<u>Saturated zone:</u>
- depth of tertiary system;
- drainage resistance of tertiary system (ditches and subsurface drains);
- drainage resistance of secondary system (major water courses);
- saturated hydraulic conductivity;

Unsaturated zone:

- physical soil properties (one typical soil unit for entire area);
- root zone depth constant for all land uses;
- water retention characteristics reduced by 15% for zero pressure head (no reduction for −1.0 m pressure head).

Preliminary calculations suggested that a variation of the boundary roughness coefficient k_M for the water courses had a very minor effect on the groundwater levels and discharges. Therefore this parameter was not changed in the sensitivity analysis. The transmissivity (T) actually required in the model depends on the water-bearing part (Eq. 41) and is updated each time step. Therefore the saturated hydraulic conductivity was varied in the sensitivity analysis.

The results of the sensitivity analysis for the saturated zone are given in Table 11. In general the effect of changing the drainage resistance and the depth of the tertiary system (including subsurface drains) has little effect on the groundwater levels. For the discharges the standard deviation diminishes drastically when the drainage resistance is decreased; the same applies if the depth of the tertiary system is reduced. The saturated hydraulic conductivity for the phreatic aquifer has different effects on different parts of the catchment. For some nodes an improvement is obtained by an increase in conductivity. A low conductivity improves the higher discharges. In Table 11 the peak flow (11 March 1981) and the base flow (middle of August) are also given. Heavy rainfall in March results in a measured peak flow of 12.6 $mm \cdot d^{-1}$. The calculated peak is higher than measured for a drainage resistance of 60 days and for a hydraulic conductivity of 1 $m \cdot d^{-1}$. In the other cases the calculated peak is lower.

The unsaturated zone parameters used in the sensitivity analysis give the results shown in Table 12. Whether one physical soil unit (no. 3 or 5) is considered does not change the results appreciably. The same applies for the thickness of the root zone. Reducing the retention characteristics of the unsaturated zone improves the discharges considerably, but the groundwater levels improve in only part of the catchment. The peak flows increase and the base flow decreases (Table 12b).

The schematization into a number of subregions based on certain hydrological conditions as discussed in Section 4.3.1, is a matter of 'professional judgement'. The main question is how many subregions should be used in the schematization. Therefore calculations were carried out with a dense schematization of subregions (71 and 81 subregions instead of 61), and fewer subregions (41 and 31 subregions). The mean standard deviation of the groundwater levels changed marginally, on average by 0.02 m when fewer subregions were considered. Increasing the number

Table 11 *Change in standard deviation for (a) groundwater levels and (b) discharges, as part of the sensitivity analysis of saturated zone parameters. For reference simulation the standard mean deviation is taken from Table 6 (met. year 1981; location of nodes Fig. 36).*

a) Groundwater levels (cm)

Nodal point	Reference* simulation	Depth of tertiary system (m)			Drainage resistance (d)					Hydr. conductivity ($m \cdot d^{-1}$)		
		0.55	0.75	0.85	Tertiary system 60	90	Secondary system 300	400	700	1	5	20
49	31	0	2	2	1	1	4	2	-1	26	-1	21
146	15	2	0	0	0	1	5	1	5	7	2	4
264	20	-4	6	13	2	-1	1	0	0	0	0	0
177	41	6	-5	-11	-2	2	-2	0	2	-1	5	-10
53	11	1	3	7	1	0	1	1	0	7	2	10
all	22	1	1	3	0	0	1	0	0	4	0	5

b) Discharges ($mm \cdot d^{-1}$)

Amount of discharge	Reference* simulation	Depth of tertiary system (m)			Drainage resistance (d)					Hydr. conductivity ($m \cdot d^{-1}$)		
		0.55	0.75	0.85	Tertiary system 60	90	Secondary system 300	400	700	1	5	20
> 2.0	1.98	-0.14	0.08	0.10	-0.60	0.19	0.10	0.05	-0.08	-0.22	-0.11	0.17
< 2.0	0.30	-0.03	0.02	0.04	-0.08	0.04	0.01	0.01	0.00	0.07	0.02	0.00
peak**	12.0	0.8	-3.4	-4.5	3.2	-2.2	-1.3	-0.6	0.8	1.0	0.4	-1.3
base***	0.08	0.0	-0.02	-0.03	-0.02	0.0	-0.03	-0.02	0.0	-0.07	-0.05	0.03

* Reference simulation data: depth of tert. system = 0.65 m; drainage resist. tert. = 75 d, second. = 500 d; hydr. cond. = 10 $m \cdot d^{-1}$

** observed peak flow for 11 March 1981 is 12.6 $mm \cdot d^{-1}$

*** observed base flow for ten days in Aug 1981 is 0.05 $mm \cdot d^{-1}$

of subregions does not decrease the standard deviations for the nine observation points. For the discharge, fewer subregions increase the standard deviation by 4%, but increasing the subregions improves the results by reducing the standard deviation by 5%.

5.5 Validation

The model analysis as discussed in Section 5.4 was carried out using the meteorological data of 1981. From the computer experiments I concluded that preferential flow and the interception by the subsurface drains considerably improved the simu-

Table 12 *Change in standard deviation for (a) groundwater levels and (b) discharges, as part of the sensitivity analysis of unsaturated zone parameters. For the reference simulation the standard mean deviation is taken from Table 6 (met. year 1981; location of nodal points see Fig. 36).*

a) Groundwater levels (cm)

Nodal points	Reference simulation	Soil unit		Thickness root zone (m)			Reduced pF*
		3	5	0.20	0.25	0.35	
49	31	2	-3	1	1	-1	6
146	15	1	6	0	0	2	-4
264	20	0	2	0	0	-1	6
177	41	-1	3	0	0	2	-7
53	11	1	5	0	0	1	7
all	22	0	2	0	0	0	2

b) Discharges ($mm \cdot d^{-1}$)

Amount of discharge	Reference simulation	Soil unit		Thickness root zone (m)			Reduced pF*
		3	5	0.20	0.25	0.35	
> 2.0	1.98	-0.01	0.14	-0.01	-0.01	0.09	-0.57
< 2.0	0.30	0.0	0.01	0.0	0.0	0.02	-0.02
peak**	12.0	0.0	-0.3	0.1	0.1	-0.6	2.5
base***	0.08	-0.02	0.02	-0.02	-0.02	0.0	-0.05

* water retention characteristics reduced by 15% for zero pressure head (no reduction for -1.0 m head)

** observed peak flow for 11 March 1981 is 12.6 $mm \cdot d^{-1}$

*** observed base flow for ten days in Aug 1981 is 0.05 $mm \cdot d^{-1}$

lated hydrograph. For the groundwater levels a small improvement was obtained by including hysteresis in the unsaturated zone. The verification period extended over the years 1982 up to 1985. Table 13 gives the standard deviations for the nine observation wells, using the input data set from Table 5. In each year of the validation period, five observation points always have a standard deviation which is less than 20 cm. The maximum deviation is 35 cm for node 49 in 1985 and the minimum deviation is 9 cm for node 52 in 1984. The average deviation for the nine observation points remains about the same in the validation period.

The mean standard deviation between the observed and simulated hydrographs are given in Table 14. For the peak flows (> 2 $mm \cdot d^{-1}$) the deviation for the reference simulations fluctuates slightly, depending on the meteorological conditions. The deviation for 1985 is always greater than for the other years. This year is characterized by several wet periods during the summer period. Introducing the preferential flow and surface runoff improves the calculated results for all the years. These processes are therefore essential for the modelling of discharges in a small catchment, such as the Hupselse Beek. The deviation for the low flows (< 2 $mm \cdot d^{-1}$) remains about the same, even when the different processes in the unsaturated zone are considered.

Table 13 *Mean standard deviation of groundwater levels (cm) for period 1982 up to 1985 for validation of MOGROW model. For the reference simulation (1981) the mean standard deviation is given from Table 6 (location of nodal points see Fig. 36).*

Nodal point	Reference simulation	Validation period				Entire period
	1981	1982	1983	1984	1985	1981-1985
49	31	23	11	11	35	22
146	15	22	31	28	15	25
166	25	16	21	25	18	21
264	20	21	21	19	25	21
177	41	24	29	29	14	27
156	19	14	20	22	32	21
130	12	12	15	11	14	13
52	19	16	11	9	13	14
53	11	15	15	10	26	16
all	22	19	20	18	22	20

Table 14 *Mean standard deviation for (a) peak flows ($mm \cdot d^{-1}$) and (b) low flows ($mm \cdot d^{-1}$) for period 1981 up to 1985, using different processes in the unsaturated zone.*
ref - Reference simulation (data set of Table 5)
I - Preferential flow and interception by drains (assumptions given in Table 9)
II - As I plus surface runoff (assumptions given in Table 9)
III - As II plus hysteresis (assumptions given in Table 9)

a) peak flows (> 2 $mm \cdot d^{-1}$)

Run	1981	1982	1983	1984	1985
ref	1.98	1.75	1.84	2.11	2.37
I	1.92	1.72	1.81	2.04	2.32
II	1.74	1.71	1.73	1.95	2.24
III	1.71	1.70	1.69	1.86	2.23

b) low flows (< 2 $mm \cdot d^{-1}$)

Run	1981	1982	1983	1984	1985
ref	0.31	0.22	0.24	0.26	0.43
I	0.29	0.22	0.24	0.26	0.42
II	0.30	0.24	0.25	0.27	0.40
III	0.30	0.24	0.25	0.26	0.38

5.6 Evaluation of the hydrological model

From the foregoing it is clear that I succeeded in integrating the groundwater model SIMGRO and the surface water model SIMWAT into one hydrological modelling system. I also demonstrated that groundwater levels and discharges could be successfully modelled over long periods with this integrated model. The computational time was not excessive. In a small catchment, such as the Hupselse Beek area, groundwater levels and discharges respond very quickly. The computed groundwater levels compare well with the observed levels. Varying certain parameters as part of the sensitivity analysis has little effect on the calculated groundwater levels. The main problem not yet solved in this respect is the comparison of observed levels for one location and the calculated average levels for a nodal point (Fig. 39). This difference in observed and calculated level becomes more pronounced, depending on the location of the observation well.

The simulation of the outflow hydrograph is very sensitive to variations in the input data and to the processes considered in the unsaturated zone. For an accurate simulation of a single rainfall event the time step used in the groundwater module should be about 3 hours (Fig. 48). In a small catchment with a direct response, as in Hupsel, this small time step is necessary. The pseudo-steady state approach used to model the unsaturated flow is not actually suitable for such small time steps. A time step of one day is sufficient when simulating the daily discharges. In larger catchments the magnitude of the time step is probably less important.

For the simulation of a single rainfall event, it was necessary to start the calculations well before the actual period. The conditions prior to the event (e.g. the retention of rainfall within the catchment) greatly influence the results. Using only a small calculation period, the initial groundwater levels and moisture storage in the unsaturated zone will control the results. For instance by changing the assumed groundwater levels at the start of such a small calculation period, an outflow hydrograph which matches the observed hydrograph can easily be produced (see for instance BATHURST, 1986).

The interflow process included in the model gave a smaller standard deviation, but the peak discharges were underestimated. The interflow process results in a direct response to rainfall, so the high discharge was therefore averaged out over a longer period.

The peak flows occurring in summer, when the groundwater levels are generally below the bed of the ditches, indicates that surface runoff and the interception by the subsurface drains can occur. The surface runoff, assumed as 2-3% contributes to some extent to the peak flows, but the interception by the subsurface drains is much larger, and results in the simulated peak flows being in the same order as the observed peak flows.

The preferential flow as used in the model was a try-out, but it will be necessary to improve the schematization concept. A way of adjusting the soil water content and the unsaturated hydraulic conductivity to the volume fraction of soil occupied by preferential flow paths has been reported recently by VAN DAM et al. (1990). In that case a prescribed height of the phreatic surface is required, but this is not possible in a regional model. The approach adopted should preserve a closed water balance in time. The irregular wetting of the soil profile can be modelled by means of two adjacent one-dimensional unsaturated zone models, one of which represents the soil occupied by preferential flow paths (wet patches) and the other the stagnant parts (Fig. 49). The precipitation is divided over the two columns, with the wet part receiving most of the rainfall. The areal proportion of each column can vary in time, depending on wet and dry periods and also in relation to the moisture conditions in the upper part of the soil. Other aspects as discussed in Section 4.2.2, such as land use and soil type, will influence this proportion as well. Further research is necessary to assess the temporal division into dry and wet patches. The preferential flow influences the fluctuation in groundwater levels. After heavy rainfall in summer the preferential flow will give a direct change in groundwater levels. This rise in groundwater level is enhanced because of the smaller storage capacity of the unsaturated zone caused by the stagnant part. The reduction in storage capacity of the unsaturated zone was also achieved by reducing the retention characteristics of this zone (pF curve shown in Fig. 22).

The number of subregions used to model the unsaturated zone flow did not greatly affect the results. The groundwater levels and discharges calculated did not change much, not even when the number of subregions was halved. Because the drainage and hence the direct runoff in Hupsel responds very quickly, these refinements were less important.

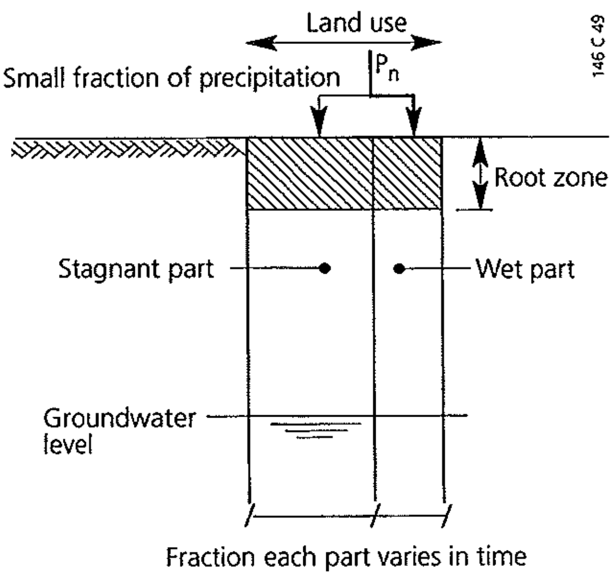


Fig. 49 *Proposed schematization of unsaturated zone for simulation of preferential flow. Each land use is divided up in a wet part (preferential flow) and a stagnant part.*

6 Application of the model MOGROW in the Poelsbeek and Bolscherbeek catchments

The MOGROW model was applied to the Poelsbeek and Bolscherbeek catchments. The aspects related to the schematization of the area, the input data required and the calibration of these data are covered in this chapter. The next step will be the case study (see Chapter 8). The weed growth in the water courses and the estimation of the flow resistance will be discussed in Chapter 7.

6.1 Description of pilot area

The Poelsbeek and Bolscherbeek catchments are located in the east of the Netherlands near Enschede (Fig. 50). The Poelsbeek catchment is approximately 41 km^2 and the Bolscherbeek 23 km^2. The two brooks originate near the town of Haaksbergen and flow north-west to join the Twentekanaal. The ground surface slopes from about 30 m above NAP (reference level in the Netherlands) in the south-east to about 12 m above NAP in the north-west. The land use is predominantly pasture, silage maize and woodland.

Both the Poelsbeek and Bolscherbeek were originally part of the catchment of the Overijsselse Vecht. A tributary of this river, the Regge, had its major ‘source’ of water near Haaksbergen. Before 1870 no major human interference in the water management occurred, the brooks often changed their courses in response to natural conditions. From 1880 onwards several water boards were established in the region, many water courses were constructed and many brooks were canalized. This construction and containment works drastically influenced the catchment hydrology. The digging of a shipping canal from Zutphen to Enschede (Twentekanaal) in 1932 caused another change in the hydrology of the region. The Poelsbeek and Bolscherbeek were among the brooks linked directly with this canal to relieve the river Regge in the event of high floods. The new navigation route also affected the groundwater levels in the surrounding area. However, within the research area this influence has been marginal. The water level in the Twentekanaal (10.0 m above NAP) is approximately 1.5 to 2.0 m below the adjacent land surface. At present the former catchments of the river Regge together with the river Dinkel are controlled by a single water board: Regge en Dinkel. Over an area of 1350 km^2 the water board is responsible for managing water quantity and quality (WATERSCHAP REGGE EN DINKEL, 1988).

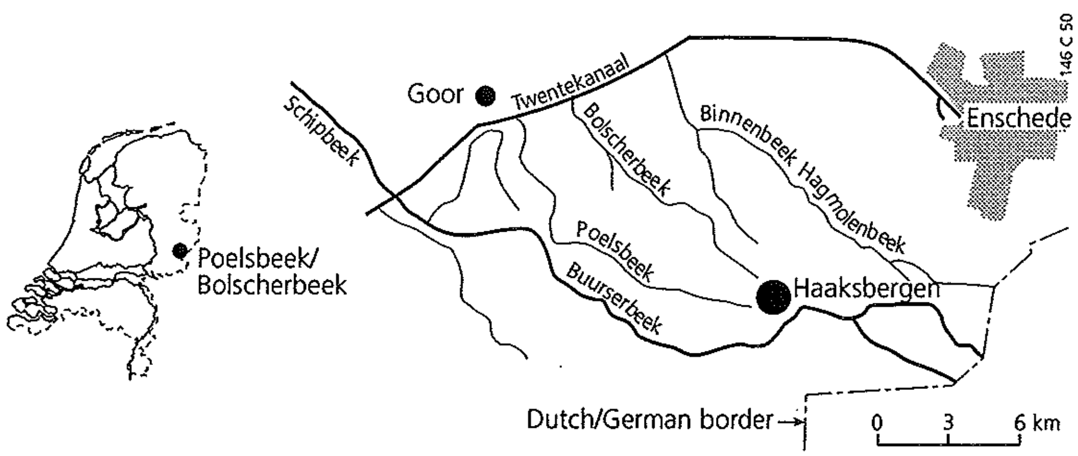


Fig. 50 Location of the Poelsbeek and Bolscherbeek in the eastern part of the Netherlands. Both water courses flow into the Twentekanaal.

The BODEMKAART VAN NEDERLAND (1979) provides information on the subsurface. The area west of Haaksbergen (Fig. 50) consists of middle Trias and marine clays from tertiary formations, which are overlain by more recent deposits. East of Haaksbergen the older deposits are very shallow or are on the surface. In the Saalien period the area was covered with land ice, which left ice-pushed ridges running north-south, and some glacial material (Drente Formation). During the Weichselien a thick layer of cover sand was deposited. Sand and clays were deposited during the Holocene period. The region with cover sands was mainly bare land, with small patches used for pasture and arable land (near Haaksbergen). This was the traditional farming, using the manure of the livestock as fertilizer. When chemical fertilizers were introduced, the remaining bare land was reclaimed (from 1880 onwards).

The network of water courses in the research area is shown in Figure 51. Approximately 83 km (20 $m \cdot ha^{-1}$) of water courses in the Poelsbeek catchment are managed by the water board, and 45 km (20 $m \cdot ha^{-1}$) in the Bolscherbeek catchment. The difference in height of about 18 metres from south-east to north-west means that a number of weirs are needed to control water level and flow. Most of the weirs are adjustable, so that the target water level in summer can be raised. The Poelsbeek mainly drains agricultural land and the discharge is very irregular. In summer the channel is nearly dry and in winter the average discharge is about 0.3 $m^3 \cdot s^{-1}$. The Bolscherbeek has a minimum discharge in summer of about 0.1 $m^3 \cdot s^{-1}$ because of the continuous discharge of effluent from two sewage treatment plants (Fig. 51). The dimensions of the water courses are based on a specific (design) discharge of 1.2 $l \cdot s^{-1} \cdot ha^{-1}$ (10.4 $mm \cdot d^{-1}$). The dimensions are such that the expected High Water level is about 0.60 m below the land surface (WERKGROEP AFVOERBEREKENINGEN, 1979). This High Water level is reached or exceeded 1 to 2 days

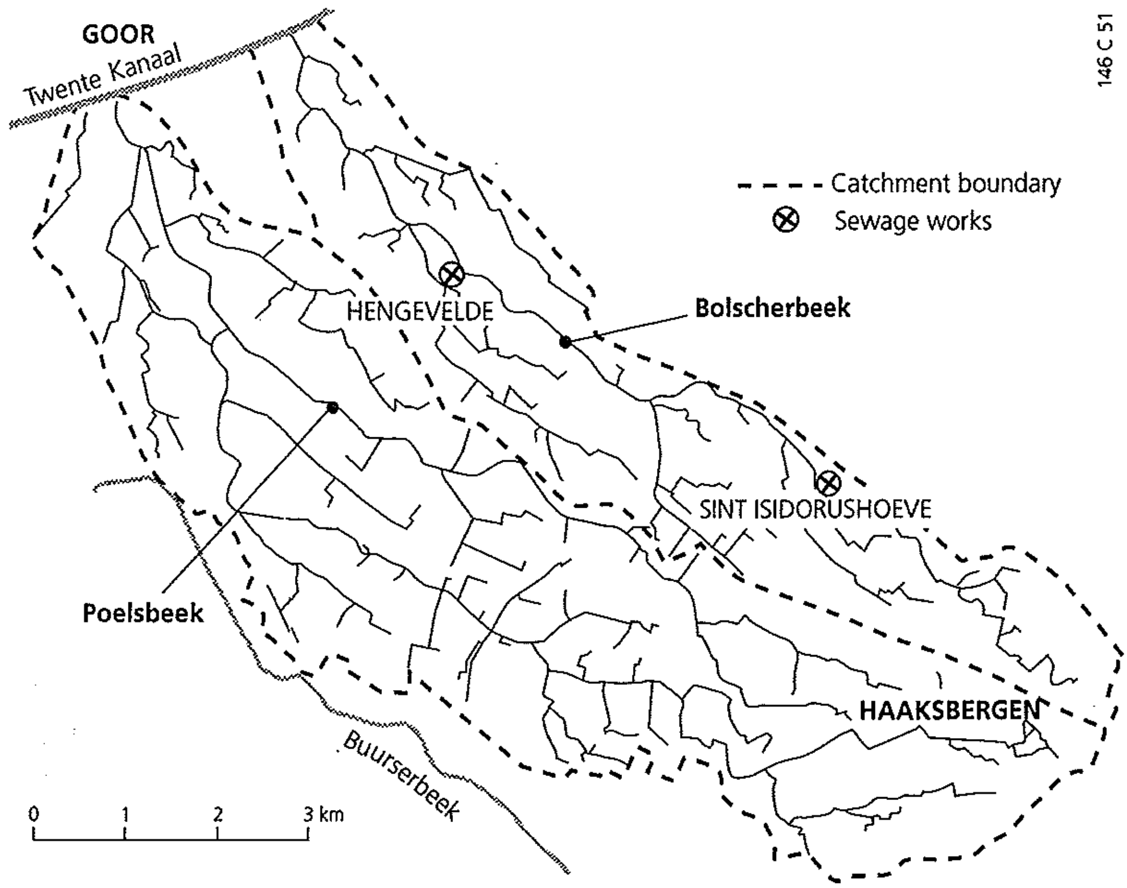


Fig. 51 *Water courses of the Poelsbeek and Bolscherbeek catchments controlled by the water board Regge and Dinkel. The two sewage treatment plants, discharging the effluent into the Bolscherbeek, are indicated.*

a year. A flow resistance of k_M=35 $m^{1/3}{\cdot}s^{-1}$ has been used for the design of the larger water courses (class 5 of Table 17). The value for the smaller water courses (class 3 and 4 of Table 17) depends on the water depth as given by Equation (7).

The water board does some weed control and contracts out the remainder. In the research area the major water courses have access roads, i.e. a maintenance path 1.5 m wide on either side. In general, the weed is cleared mechanically three times a year. The first clearance starts the beginning of May, the second is in June and the last is before winter. If the weather in spring and early summer discourages weed growth, two clearings a year are adequate. Generally, weed is cleared by means of a tractor-mounted cutter bar, with a rake that deposits the weeds on the access road. The banks and a small portion of the bed can be cleared in this way. A weed-cutting bucket attached to a tractor is used for the larger water courses. The cutting bucket is able to cut weeds on both banks and on the bed of the water course. The weed growth for the Poelsbeek and Bolscherbeek catchments over time and its relation with the flow resistance are discussed in Sections 7.1 and 7.2.

6.2 Input data

The same type of schematization applies for the hydrological modelling as was used for the Hupsel application (Section 5.2). The finite element network for the saturated groundwater flow consists of 787 elements with 437 nodes. Each nodal point represents approximately 25 ha (node spacing 500 m). Based on the locations of weirs, subcatchments, variation in depth of the groundwater and the physical soil properties, 78 subregions were identified (Fig. 52). The major water courses were considered to be part of the surface water network. The remaining ditches were treated as reservoirs and connected to the network (Section 4.3; Fig. 32). For the SIMWAT module, 151 sections were used, each representing a certain stretch of water course and also 61 weirs. The dimensions and invert levels of the water courses were obtained from the water board. In the larger water courses (class 5 of

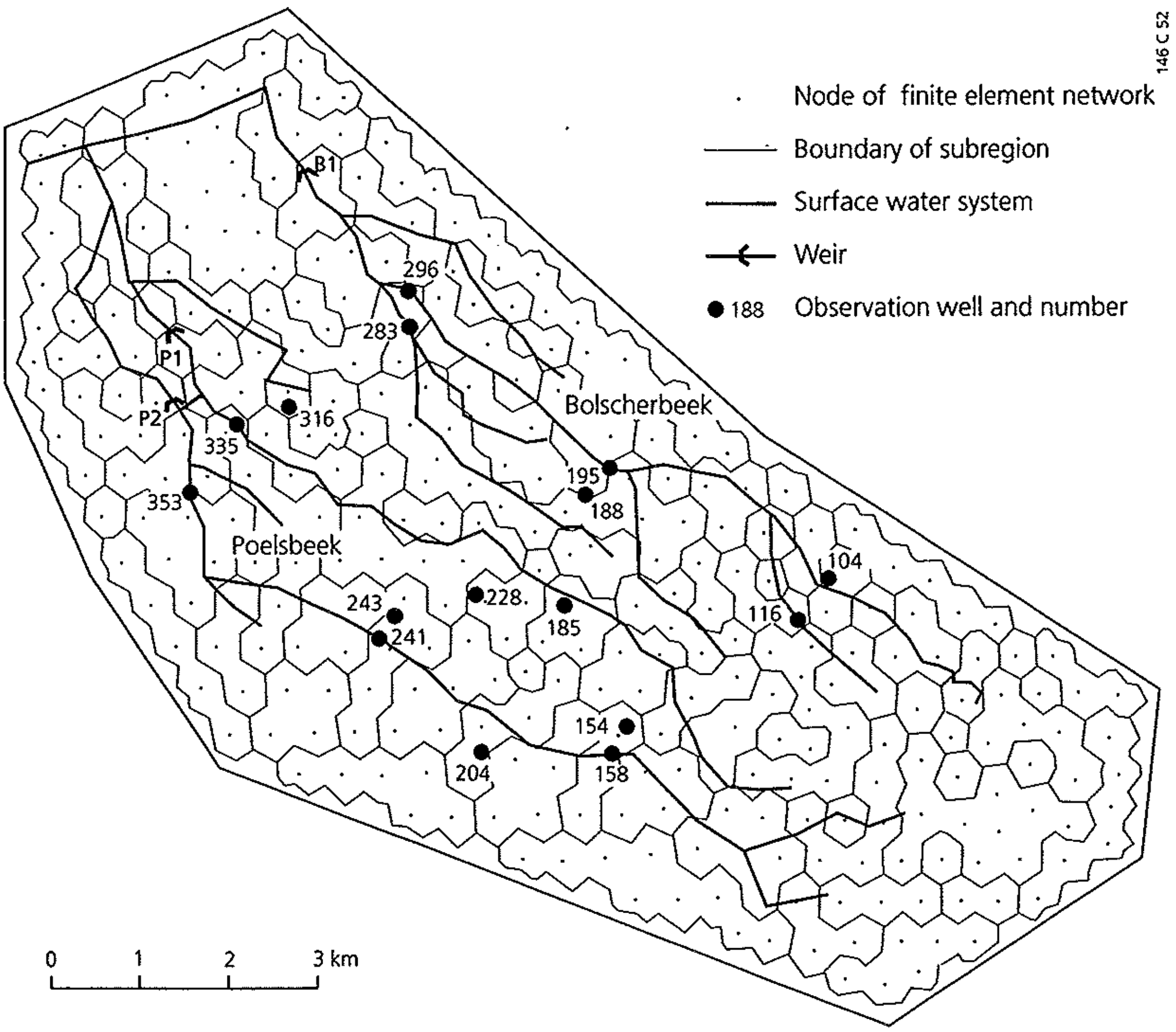


Fig. 52 The schematization of the groundwater system in a number of subregions. The water courses and weirs are represented by sections between two nodes. The location and identification numbers of observation wells and gauging stations are also shown.

Table 17) the culverts or bridges present are so large that their influence on the flow regime is negligible. Quite a few culverts are present in the smaller water courses. These have a major effect on the flow rate, caused by their back water effect. Instead of considering all the culverts in the model, an equivalent flow resistance was used to account for the effect of the culverts on the discharge capacity. This flow resistance was calculated by the programme CULV, which was developed to evaluate the effect of a number of culverts on the flow resistance of a channel section (BABOS, 1990). The program uses the direct step method (CHOW, 1959) to calculate the surface water profile in a section with a number of culverts. An equivalent roughness coefficient, k_M^e, is then determined, using the total head loss for a section. The different sizes and the number of culverts found in the field made it impossible to consider each situation separately. The equivalent roughness coefficient, k_M^e, was calculated for a typical cross-section and a number of culverts (Fig. 53). I assumed that the size of the water courses was related to the size of the culverts present and the back water effect they might cause. Based on results given in Figure 53 I assumed that in water courses with a bed less than 1.0 m wide the roughness coefficient was 25 $m^{1/3} \cdot s^{-1}$, and if the bed was between 1.0 and 1.5 m wide it was 28 $m^{1/3} \cdot s^{-1}$. In the larger water courses no back water effect from the culverts was noticed and the roughness coefficient was taken as 32 $m^{1/3} \cdot s^{-1}$ (see Fig. 70b). This value applies to the unobstructed part of the cross-section. The reason for using only the unobstructed part is discussed in detail in Section 7.2.

In the Poelsbeek and Bolscherbeek catchments one phreatic aquifer is present (GRONDWATERPLAN OVERIJSSEL, 1986). The saturated hydraulic conductivity of this aquifer is in the range 12 to 14 $m \cdot d^{-1}$. The thickness of the aquifer varies between 10 and 60 m.

The land use which is required as an input for the hydrological modelling was obtained from a land use data bank (LGN) containing this information per grid cell of 25x25 metres for the Netherlands (THUNNISSEN et al., 1992). The data obtained from this data bank for the research area are given in Table 15.

Table 15 *Land use (as % of total area) in the Poelsbeek and Bolscherbeek catchments. Source: Land use data bank (LGN) of the Netherlands.*

Land use	Area (%)
pasture	46.8
maize	20.1
deciduous woodland	15.6
coniferous woodland	8.7
nature area (grassland)	1.2
residential area	7.6

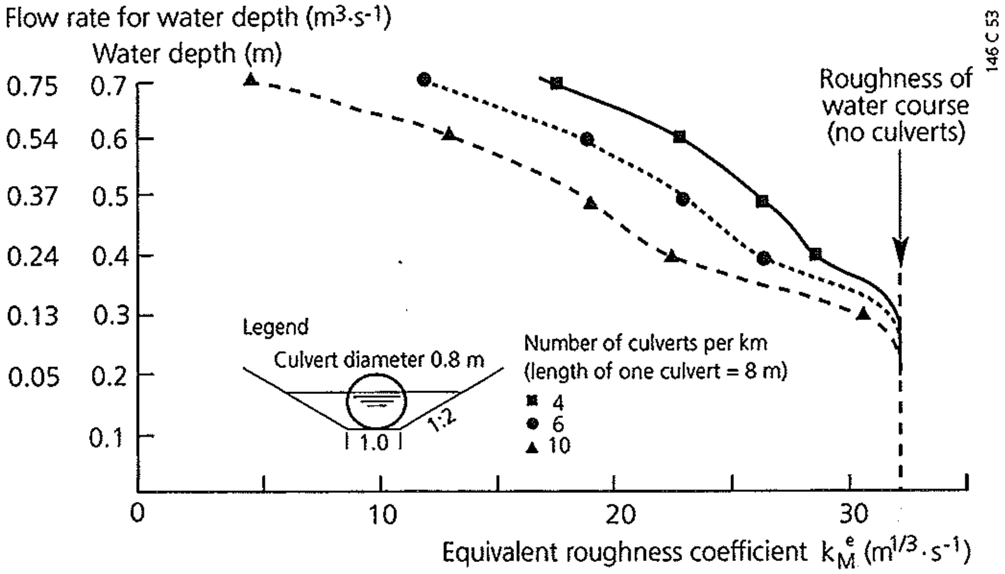


Fig. 53 *Equivalent roughness coefficient k_M^e calculated for a range in water depths and corresponding flow rates of a water course, to account for a number of culverts present in reality (culvert diameter 0.8 m).*

Soil data were available in a Geographical Information System with grid cells of 50 x 50 m (DENNEBOOM et al., 1989). Per grid cell a description of the soil is available together with a class indicating the water table fluctuations. Within the research area 30 mapping units varying from 0.01 to 27 km^2 are present. For the hydrological modelling a number of physical soil units characterized by the water retention and hydraulic conductivity were required. Generalization of the mapping units resulted in four such units. The main physical soil groups classified are all sandy soils; one is a humus podzol and the other three are sandy gley soils with some loam (Table 16). The characteristics of these soils were taken from the 'Staring series' (data bank of water retention and hydraulic conductivity characteristics of topsoils and subsoils in the Netherlands; WÖSTEN et al., 1987). The soils B1 and B2 in combination with O1 were taken. The characteristics required for the unsaturated zone modelling were calculated with the CAPSEV model (Section 4.3.4).

The interaction between groundwater and surface water needs to be characterized by a drainage resistance (Eq. 33). This resistance is derived from hydrological parameters and the spacing of the water courses. The density of the water courses controlled by the water board was obtained from the map (Fig. 51), and the smaller ditches not controlled by the water board were measured in the field (Table 17). ERNST (1978) gives drainage resistances derived from field measurements (Table 18) for an area with drainage conditions similar to those in the pilot area. Also given are the resistances calculated with the Bruggeman formula (BRUGGEMAN,

Table 16 Description and area of selected physical soil units.

Soil unit*	Area (km^2)	Typical location	Generic Dutch name
Hn21	25.4	Recent reclaimed areas, woodland and peaty fields	'Veldpodzolgronden'
EZ21	8.3	Ridges	'Enkeerdgronden'
EZ23	7.1	As above, higher loam content	'Enkeerdgronden'
pZg23	23.2	Valleys of the brooks	'Beekeerdgronden'

* most frequent or general mapping unit

1978). These authors use different parameters to obtain the drainage resistance (Table 18), but the resistances obtained do not differ much for the conditions I considered.

The regional groundwater flow follows the two brooks, i.e. is north-westwards. The flux on the boundaries was obtained from isoline maps and the transmissivity of the aquifer. The incoming flux on the south-eastern boundary was estimated as 2200 $m^3 \cdot d^{-1}$ and the outgoing flux at the north-western boundary as 900 $m^3 \cdot d^{-1}$. The variation between summer and winter was negligible.

6.3 Calibration and verification of input data

The phreatic groundwater levels from June 1989 onwards were available for 15 locations (Fig. 52). The discharge from the two catchments could not be measured at the confluence with the Twentekanaal, because the weirs at these locations are

Table 17 Density of water courses ($m \cdot ha^{-1}$) for five depth classes found in the Poelsbeek and Bolscherbeek area.

Class	Depth of water course (m)	Catchment (ha)	Poelsbeek		Bolscherbeek	
			Lower	Upper	Lower	Upper
1	0.4-0.6	< 200	53	70	48	
2	0.6-0.8	< 200	70	77	50	
3*	0.8-1.2	< 200	12	15	12	10
4*	> 1.2	200-1000	5	4	3	9
5*	> 1.5	> 1000	4	-	7	-

* Controlled by the water board Regge en Dinkel

Density ($m \cdot ha^{-1}$)	Ernst* (d)	Bruggeman** (d)
75	105	140
50	285	220
12	800	950

Table 18 *Drainage resistances obtained from ERNST (1978) and calculated with the equation given by BRUGGEMAN (1978).*

* Transmissivity T = 300 $m^2 \cdot d^{-1}$

** Aquifer thickness = 25 m; horiz. conductivity = 13 $m \cdot d^{-1}$; vert. conductivity = 6.5 $m \cdot d^{-1}$; hydr. resistance of the bed of the water course = 1.0 d

often submerged or shipping waves cause unwanted fluctuations in water level. Therefore the gauging stations were located further upstream (Fig. 52).

The input parameters were calibrated by comparing observed and calculated groundwater levels and discharges. The computations were carried out over the period June 1989 until September 1990 for the calibration and from October 1990 until September 1991 for the verification. The time step was one day for the groundwater module and 15 minutes for the surface water module. It took approximately 3 hours CPU time to simulate one year on a VAX 4200.

The drainage resistances for the interaction groundwater and surface water were initially taken from Table 18. For the channel system the hydraulic resistance of the bed was derived from field measurements. The differences in groundwater level between two locations (1 and 3 m away from the water course) were taken and related to the discharge of the catchment. For the secondary and tertiary system the resistances were calibrated to yield the values given in Table 19. The resistance of the trenches was not varied in the calibration phase. The depth of the three sub-

Table 19 *Drainage resistances obtained for the calibration period.*

Subsystem	Depth of water course	Drainage resistance per unit area	
		of land	of water course
	(m)	(d)	(d)
Trenches	0.50	80	-
Tertiary system	0.80	150	-
Secondary system	1.20	1000	-
Channel	varies*	-	1.2-1.4**

* Part of surface water model SIMWAT

** Hydraulic resistance of the bed of a water course systems as given in Table 19 were obtained from the field measurements (Table 17). The comparison between measured and calculated groundwater levels, as well as discharges gave no reason to change the transmissivity in the calibration phase.

Groundwater levels

Figure 54 gives the groundwater hydrographs for three locations, showing the typical differences between calculated and observed groundwater levels. The groundwater levels calculated for node 204 are slightly too high for the summer period and for node 228 are lower than the measured levels (Fig. 54). The mean standard

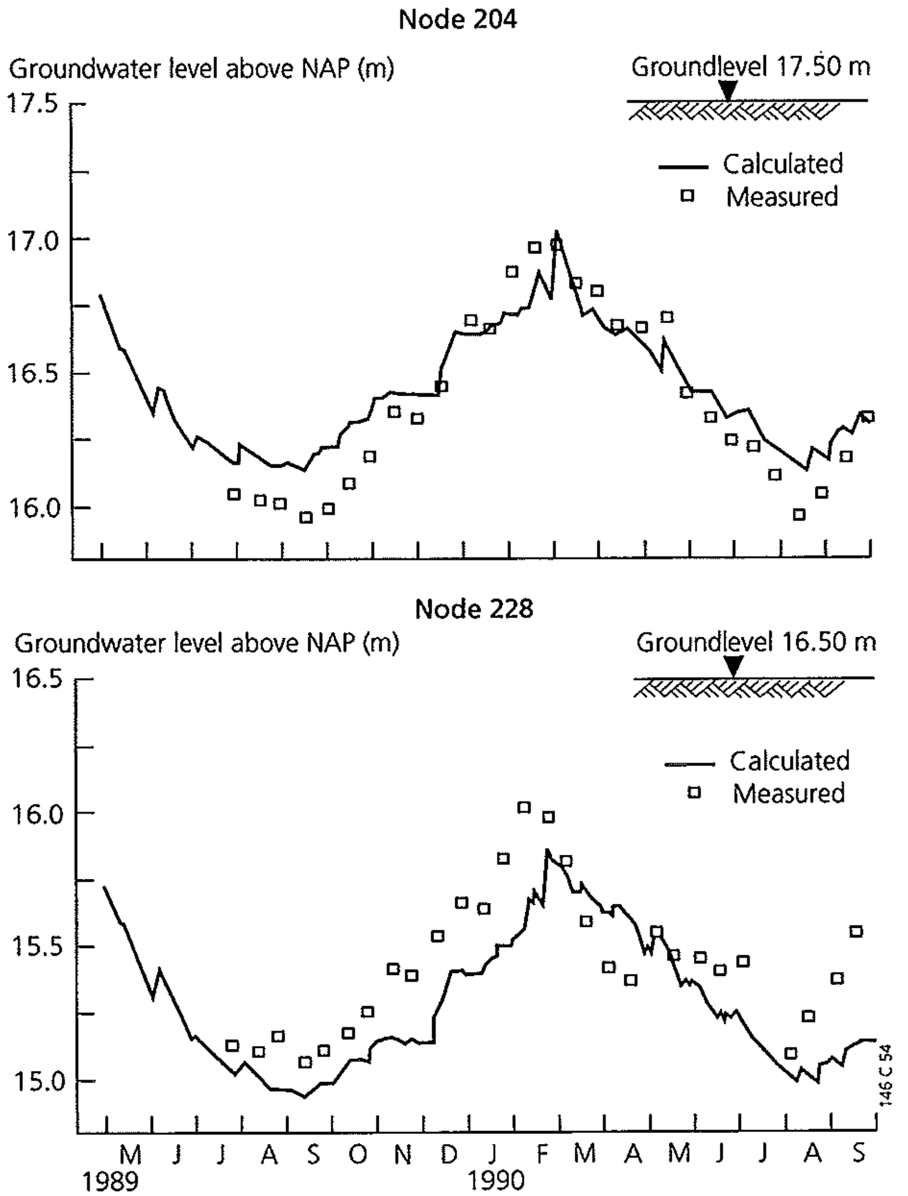


Fig. 54 Observed groundwater levels together with calculated levels for the nearest nodal points 204 and 228 (for locations see Fig. 52).

deviation (root mean square) between calculated and observed groundwater levels for the 15 locations are given in Table 20. The values are of the same order as those obtained in the Hupselse Beek catchment (Table 6). For nine locations the mean standard deviation is less than 0.20 m; only node 316 has a deviation greater than 0.30 m. The results of the verification carried out for the period October 1990 until September 1991 are also given in Table 20. The mean standard deviation for the verification period does not differ much from that for the calibration period.

Discharges

Figure 55 shows the measured and calculated hydrographs for the main stream of the Poelsbeek (weir P1). The calculated and measured discharges compare well, but small differences are noted, such as for autumn 1989, where the calculated discharge is slightly higher than observed. Some peak discharges are underestimated, which may be because special processes, such as preferential flow and surface runoff were not included in the simulations. The mean standard deviations between calculated and observed discharges at the three weirs are given in Table 21. The mean standard deviation for weir P2 is quite small compared with the discharge over weir P1, because weir P2 discharges only 10-20% of the flow (see Fig. 52 for locations of weirs). The Bolscherbeek has two locations where sewage effluent is discharged, causing an irregular flow regime and therefore the mean standard deviation is larger than for the Poelsbeek. The difference between observed and calculated discharges (Fig. 56) is reasonable. The sewer distribution network and sewage treatment works are greatly simplified in the model (Fig. 31). The results of the verification carried out for the period October 1990 until September 1991 are also given in Table 21. The deviation is slightly larger, mainly because of the differences occurring the beginning of 1991, in which there was a long period with severe frost and snowfall. The MOGROW model cannot handle such situations correctly.

Table 20 *Mean standard deviations (cm) between observed and calculated groundwater levels (calibration period: June 1989-Oct 1990 and verification period: Oct 1990-Oct 1991; see Fig. 52 for location of nodes).*

Node	Calibration	Verification	Node	Calibration	Verification
335	12	10	116	15	13
316	31	28	188	28	29
353	8	9	195	10	16
243	20	23	296	24	21
241	9	9	104	18	18
204	12	12	228	22	24
154	23	25	283	15	17
158	7	10	all	17	18

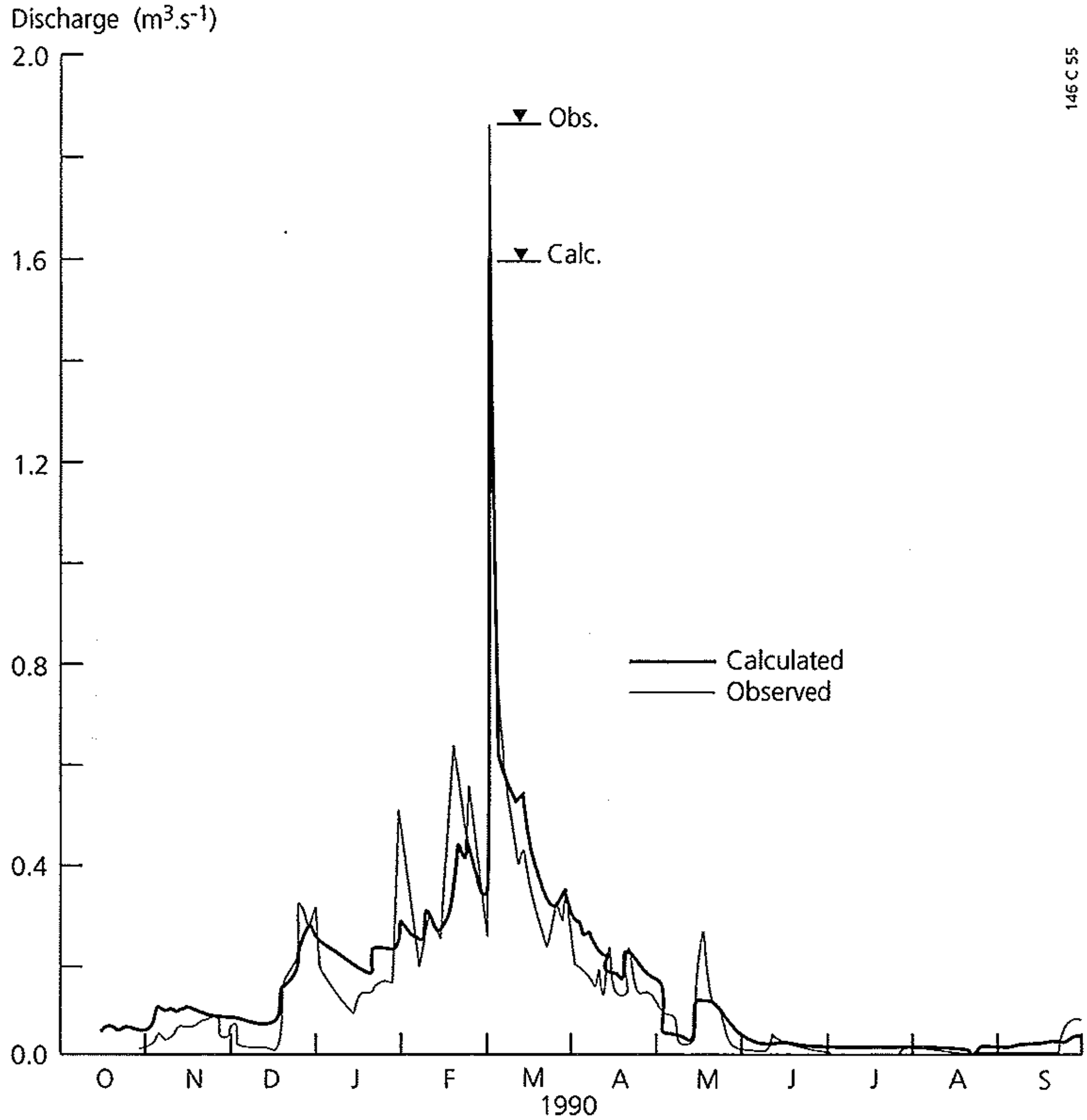


Fig. 55 *Hydrograph showing the measured and calculated discharges for the Poelsbeek at weir P1 (location of weir shown in Fig. 52).*

The standard deviation is a poor way of comparing calculated and measured discharges. It has one value for a wide range in discharges. Therefore the calculated and measured discharges were also given as a frequency of exceedance per year (Fig. 57). This enable the differences to be easily observed. The larger discharges

Table 21 *Mean standard deviation ($m^3 \cdot s^{-1}$) between observed and calculated discharges of the Poelsbeek and Bolscherbeek (calibration period: June 1989-Oct 1990 and verification period: Oct 1990-Oct 1991; see Fig. 52 for location of weirs).*

Weir	Calibration	Verification
P1	0.05	0.08
P2	0.01	0.01
B1	0.10	0.11

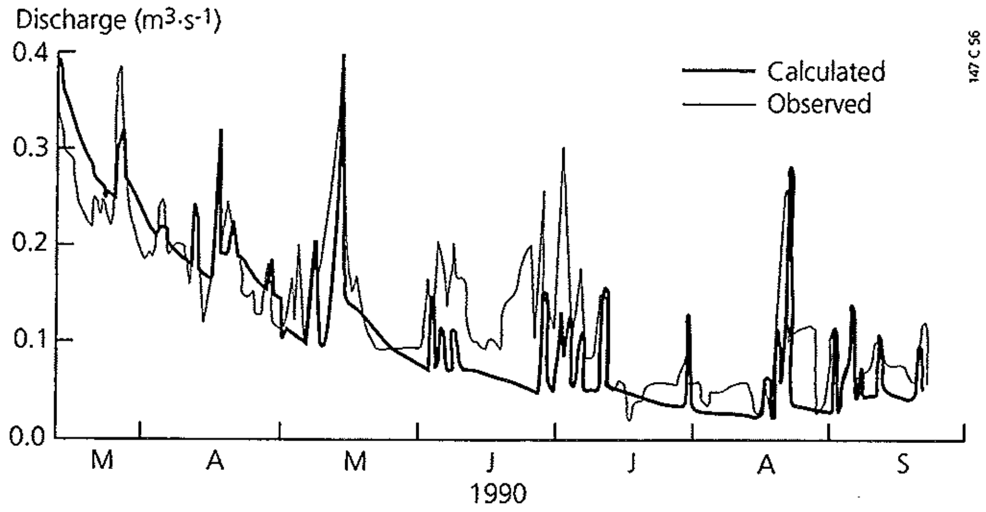


Fig. 56 *Hydrograph showing the measured and calculated discharges for the Bolscherbeek at weir B1. Two sewage treatment works influence the discharge characteristics (location of weir shown in Fig. 52).*

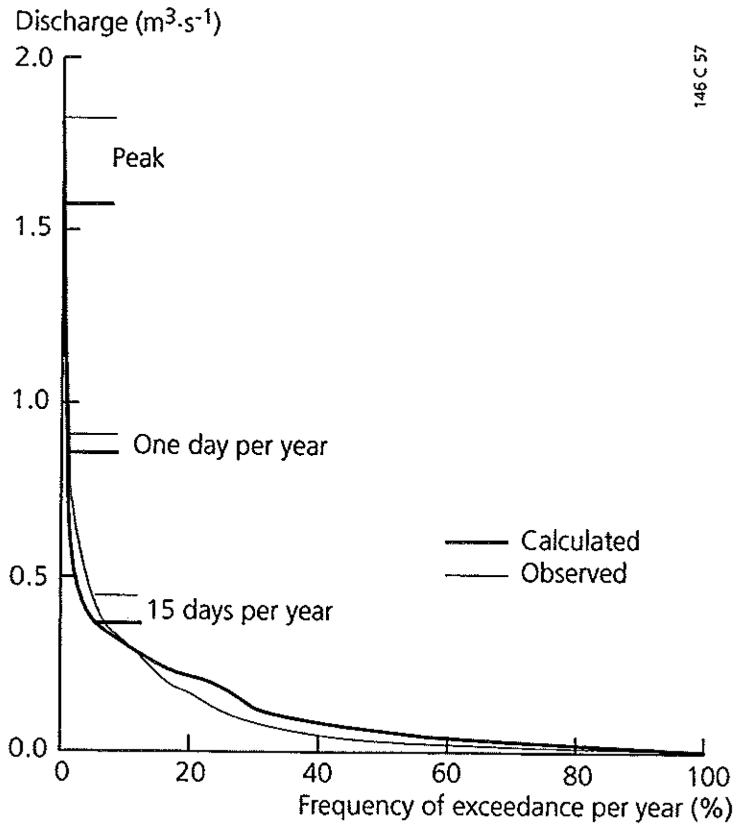


Fig. 57 *Discharge characteristics given as the frequency of exceedance per year for weir P1 of Poelsbeek, using daily calculated and observed values. Discharges occurring once and 15 times a year are indicated (calibration and verification period 1989-1991).*

occurring less than 15% of the time are slightly underestimated and the smaller flows are overestimated.

Conclusions

A detailed calibration and verification was not possible, because of the limited amount of field data available. The hydrological modelling was not as complicated as in the case of the Hupselse Beek area. In the Poelsbeek and Bolscherbeek catchments the variation in ground surface is less than in Hupsel, which results for these catchments in a higher storage capacity of the surface and groundwater system. Therefore these catchments gave discharges that react more slowly to rainfall events, but which nevertheless cannot be modelled exactly in the same way as the measured discharges. Some typical disparities between measured and calculated values were noted here too. On the other hand, reasonable agreement was obtained, even for the Bolscherbeek catchment, which has two sewage treatment works discharging effluent and occasionally an emergency overflow from the sewer system of the town of Haaksbergen.

7 Relating aquatic weed growth and flow resistance to weed control

The flow resistance as discussed in Chapter 3 and used in the flow formulae is greatly affected by aquatic plants. Water courses have to be maintained, which means removing weed growth and restoring the channel depth. Maintenance by means of weed control is an important task of the water boards.

The rate at which the cross-section of a water course fills with weeds depends on various factors. These will be discussed in Section 7.1, together with the data on weed obstruction obtained in the field. The relation between weed obstruction and the flow resistance is considered in Section 7.2. A procedure developed to estimate the timing of weed control is presented in Section 7.3.

7.1 Weed growth and the obstruction in a water course

The maximum biomass attained by aquatic weeds at a particular site is the result of the combined effects of the growth conditions (light, inorganic nutrients and carbon, water temperature and water velocity) and the plant's physiological responses at its current state of growth (DAWSON, 1988). The complex interactions of a large number of variables make it very difficult to draw general conclusions and to judge the effects of single factors. Nutrients, especially nitrogen and phosphorus, are the most important growth factors. In an aquatic environment the light available decreases down the water column. The attenuation of light (range 0.4-0.7 µm is photosynthetically active) is calculated from the Lambert-Beer law, as shown graphically in Figure 58. The extinction coefficient used in this law expresses the light penetration and depends on the amount of suspended matter. For the average light intensities during summer in the Netherlands, it has been estimated that photosynthesis is not possible when the attenuation of light intensity exceeds 93% (VERMAAT and VAN VIERSSEN, 1990). Figure 58 shows how the variation in water depth is used to define four weed growth categories, on the basis of the attenuation of light. Based on an extinction coefficient k_a=2 (VERMAAT and VAN VIERSSEN, 1990), the range for the attenuation of light intensity for category I is 0-50%, from 51-75% for category II and from 76-88% for category III. These categories reflect an amount of weed growth and are referred to in Section 7.1.1 when presenting the field data.

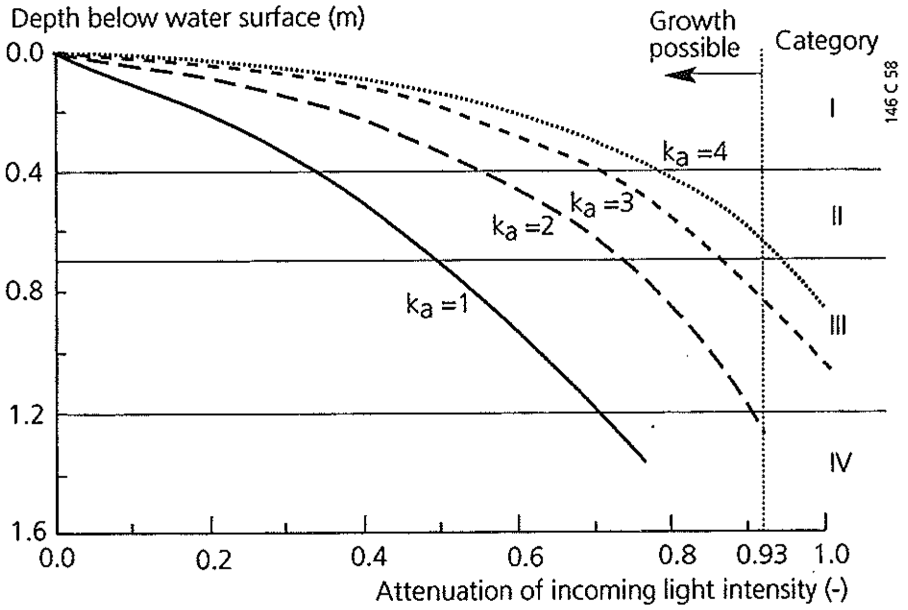


Fig. 58 *Attenuation of incoming light intensity dependent on water depth and extinction coefficient k_a. Water depth categories, based on k_a=2, are defined for similar weed growth, which reflects the reduction of incoming light intensity.*

Only a few models for aquatic weed growth are currently available, and they cover single species only. BEST (1981) developed a simulation model for Ceratophyllum demersum (hornwort). Ideally, growth models could be used to predict leaf area or biomass, which could then be translated into the obstruction in the water course. These models should be relatively simple so they can be incorporated into hydrological models. Unfortunately they are not yet available for the most common species found in the Netherlands. Therefore I used the measured extent of weed growth as the obstruction of the wetted area. I measured these obstructions in the field during the growing season, because no data over the entire growing season were available in the literature.

The high flow stage, being the maximum acceptable water level was used to estimate the maximum discharge capacity (Fig. 59). This water level means that the obstructed part often consists not only of aquatic plants, but also of plants at the water's edge, plus terrestrial vegetation. In the Netherlands the high flow stage for farming is about 0.50 to 0.60 m below soil surface and is expected to be exceeded once a year (WERKGROEP AFVOERBEREKENINGEN, 1979). I measured the relative weed obstruction, defined as the area covered by weeds divided by the wetted area, for this level (Fig. 59).

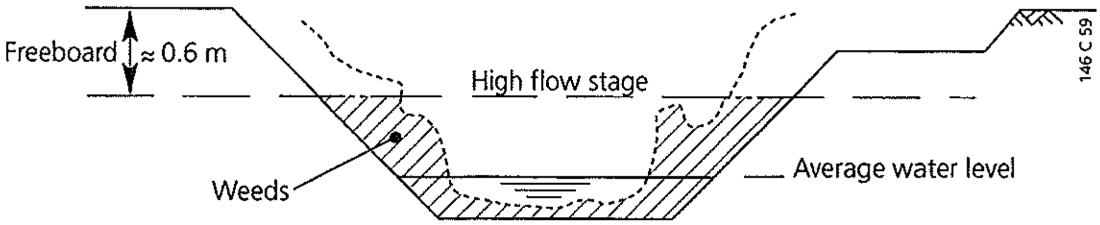


Fig. 59 *Cross-section of water course showing the high flow stage for which the relative weed obstruction (area weeds divided by wetted area) was determined.*

7.1.1 Field measurements of weed obstruction

In the Poelsbeek and Bolscherbeek catchments the weed obstructions were measured during the growing season of 1990 and 1991. The nine locations in the catchment are shown on Figure 60. Using the average water depth the nine locations were grouped according to the categories shown in Figure 58. At each location no weed removal was carried out in one stretch of the water course. At the start of the growing season the measurements were frequent (once in 10-15 days); later in

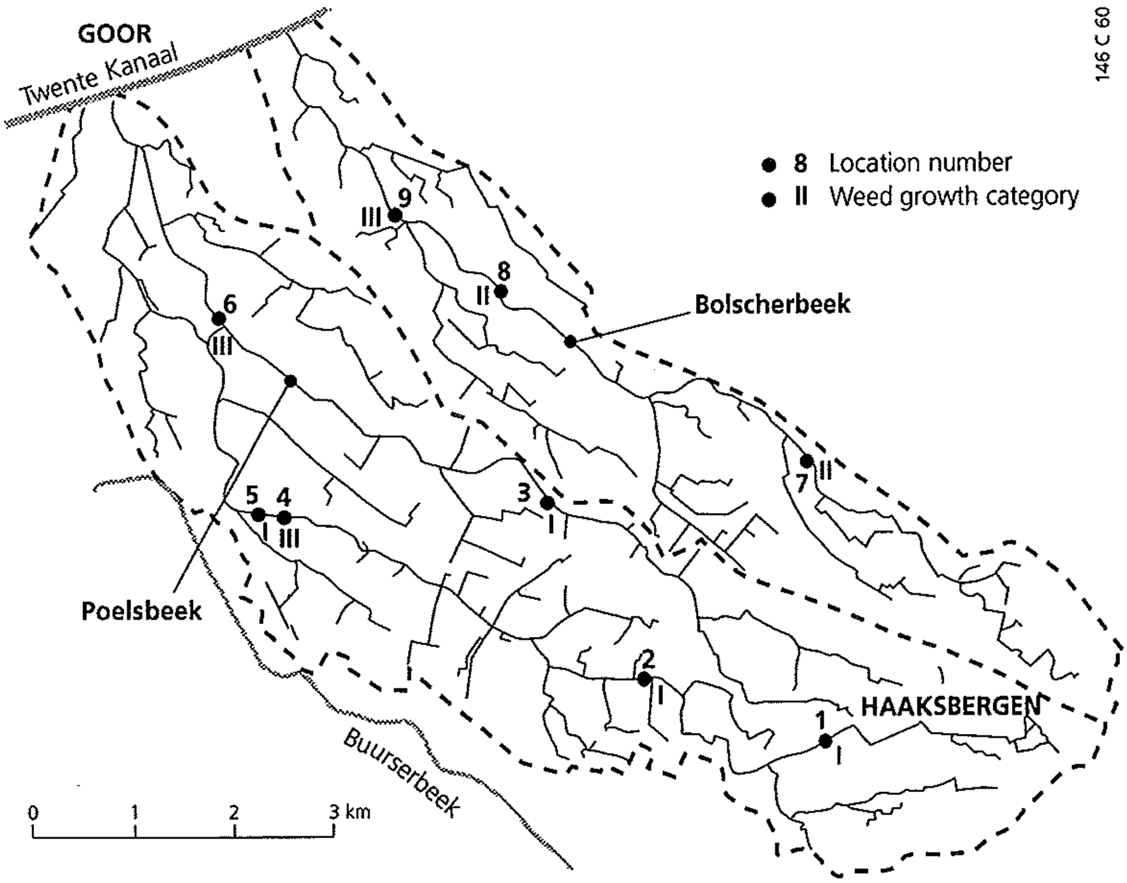


Fig. 60 *Location of water courses where weed obstruction was measured. The average water depth was used to define a category from Fig. 58.*

the year they were less frequent (once in 20-30 days). The major weed species encountered in the nine water courses are stinging nettle, common sorrel, reed sweet-grass, flote-grass, hornwort and elodea. Figure 61a gives the extent of weed growth on four dates during 1990 at location 7. As shown most of the weed growth is on the side slopes of the water course. Figure 61b shows the maximum weed obstruction during the growing season when weed was cleared by the water board. The upper ends of the weed were not included in the obstructed area, because they are easily bent over by the flowing water. The relative weed obstruction obtained for the high flow stage is shown in Figure 62, together with the bandwidth of observed data. The data shown in Figure 62 were obtained in 1990, very similar results were obtained for 1991. The increase in obstruction, as much as 35% in the period mid April to mid May, can be found in water courses from category I (Fig. 62). At some locations the adjacent land use, particularly heavily fertilized maize for a number of years, has a marked effect on this sudden increase (e.g. location 3 of category I; Fig. 62). A water course of category I reaches a maximum relative weed obstruction of about 90%, for category II this maximum is about 85% and for category III only 45%. Most of the weed growth is on the channel side slopes (Fig. 61) and therefore the relative weed obstruction for category III remains small. After mid August the obstruction decreases, because the stems bend over and start to die off. Weed clear-

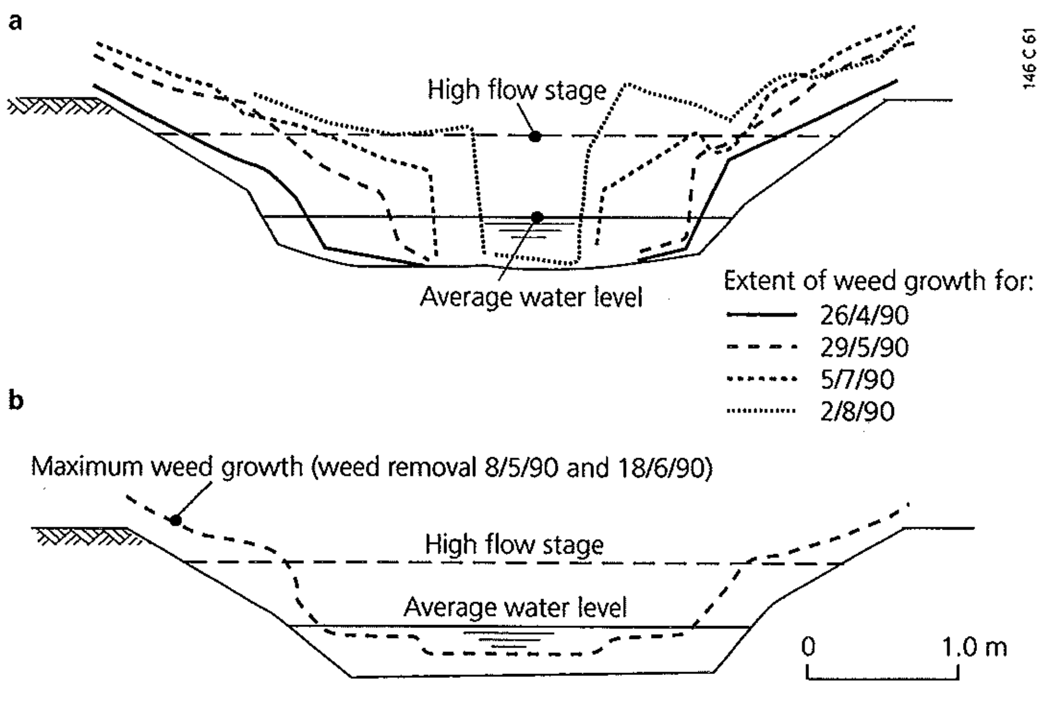


Fig. 61 *Weed growth at observation point 7 during summer 1990 (location shown in Fig. 60).*
a) Change in weed growth during the growing season (no weed clearance)
b) Maximum extent of weed growth when weed was removed twice during the growing season

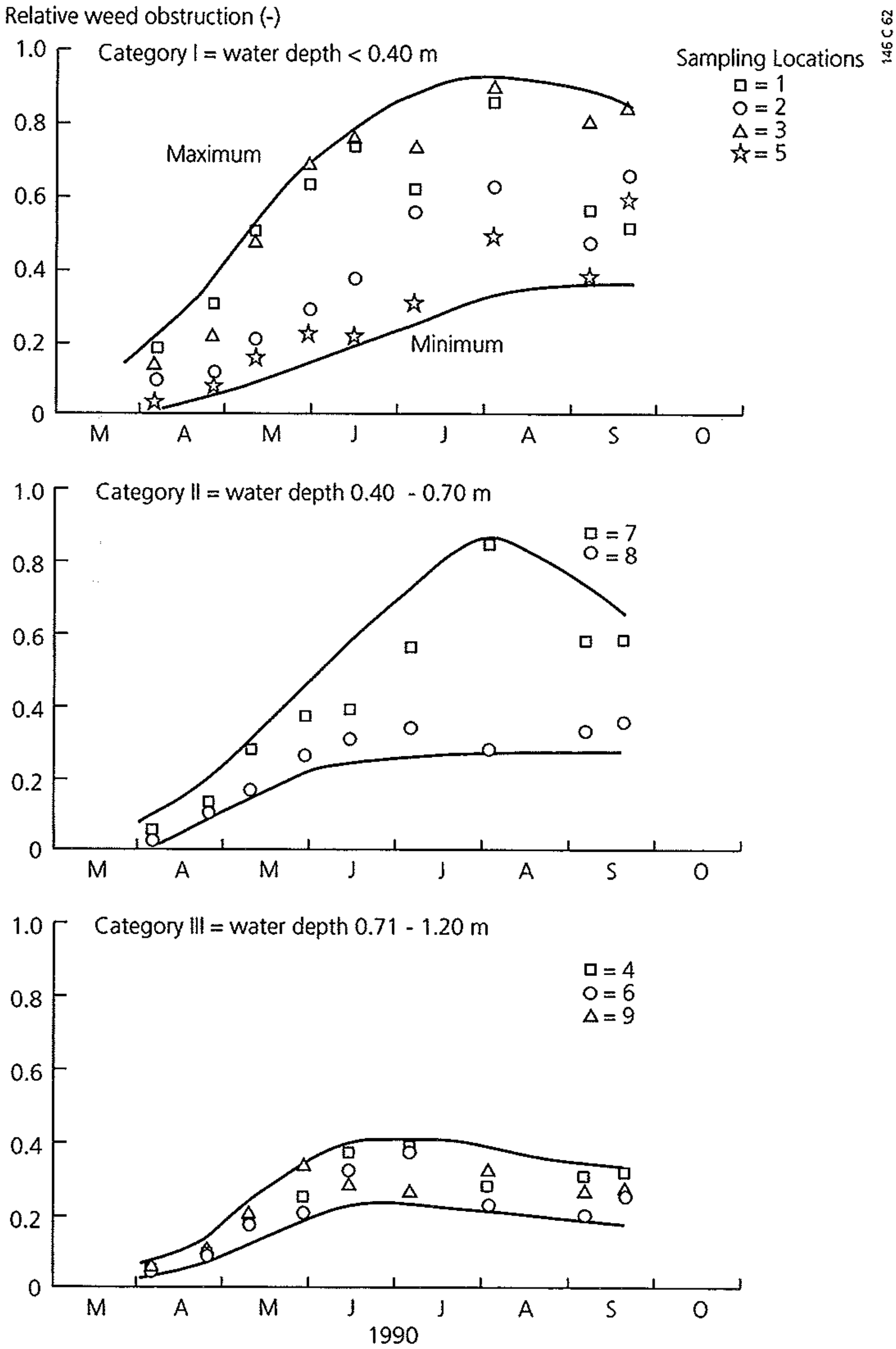


Fig. 62 Temporal range in relative weed obstruction for the high flow stage (freeboard 0.6 m) in water depths of categories I-III during 1990 (sampling locations shown in Fig. 60).

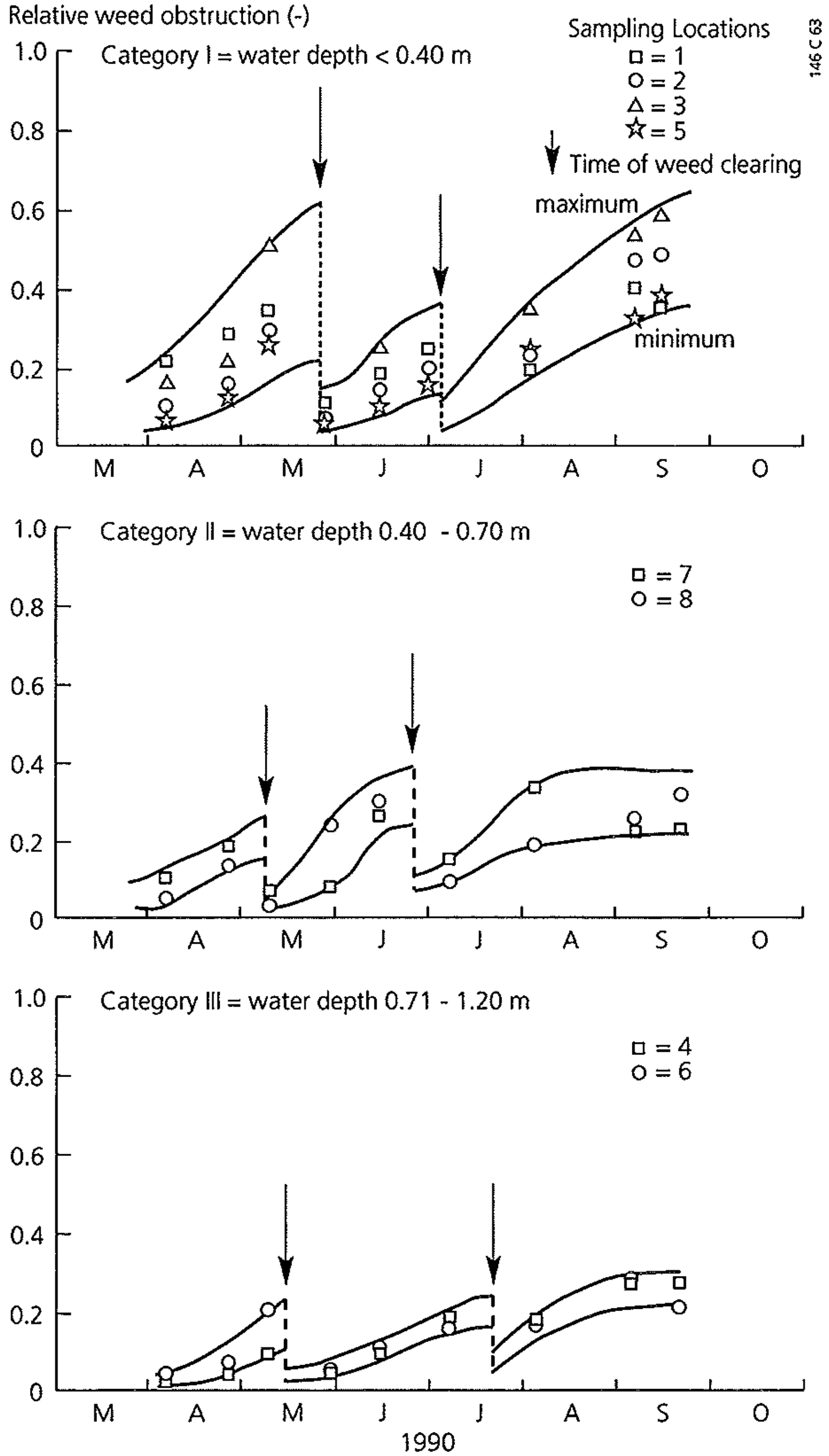


Fig. 63 *Temporal range in relative weed obstruction for the high flow stage (freeboard 0.6 m) in water depths of categories I-III in channels where weed was cleared twice during the growing season of 1990 (sampling locations shown in Fig. 60).*

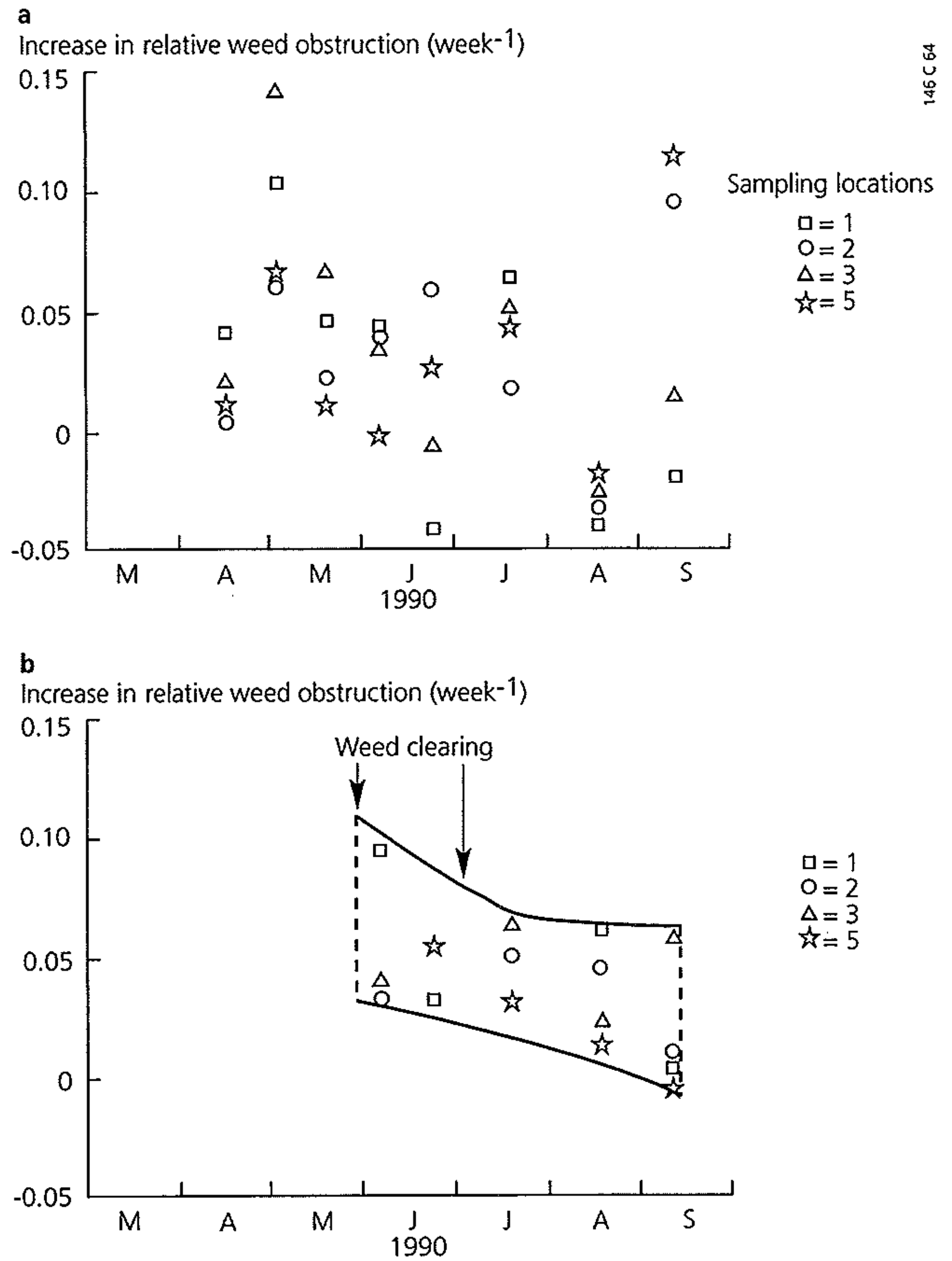


Fig. 64 *Increase in relative weed obstruction for the high flow stage (freeboard 0.6 m) in water depths of category I (water depth less than 0.40 m; sampling locations shown in Fig. 60).*
a) No clearance during growing season of 1990
b) Range after first weed clearance (end of May 1990)

ance has an effect on the relative obstruction; see Figure 63. The weed clearance results in maximum obstruction of about 60% for water depths of category I, 40% for category II and about 25% for category III. The rate at which the cross-section of category I fills with weeds is shown in Figure 64. The fluctuation in the rate is governed by the growth cycle of the different weed species (Fig. 64a). From the middle of April to the middle of May a large increase in obstruction can be noted, followed by a much smaller increase in June. These fluctuations are more pronounced in the water courses where weeds are not removed. Weed clearing in May and June, results in a regular increase in relative weed obstruction (Fig. 64b).

Some data from the literature are presented in Figure 65, for comparison with the data shown in Figure 62. The rate at which the cross-section fills remains about the same, but in Figure 65 greater maximum obstructions can be noted for category II. The weed obstruction shown in Figure 65 was measured for the average water level. The area this involves is much smaller than the area involved if the high flow stage is considered, and results in the relative obstruction being overestimated. RADERMACHER (1970) also uses the relative obstruction by weeds (Section 3.2). The results from Figure 62 compare well with his data (Fig. 8).

Measurements from more locations and a more detailed classification of the important variables that control the growth of weeds are necessary before this type of information can be used in practice. The relative weed obstruction obtained as explained above must be considered in combination with an expected discharge capacity. This combination of obstruction and expected discharge will indicate the need for weed clearance. These aspects are discussed in Section 7.3.

7.1.2 Weed obstruction in relation to flow velocity

In Section 7.1.1 weed growth was quantified in terms of relative weed obstruction. This obstruction being for a very low velocity, will be reduced when higher veloc-

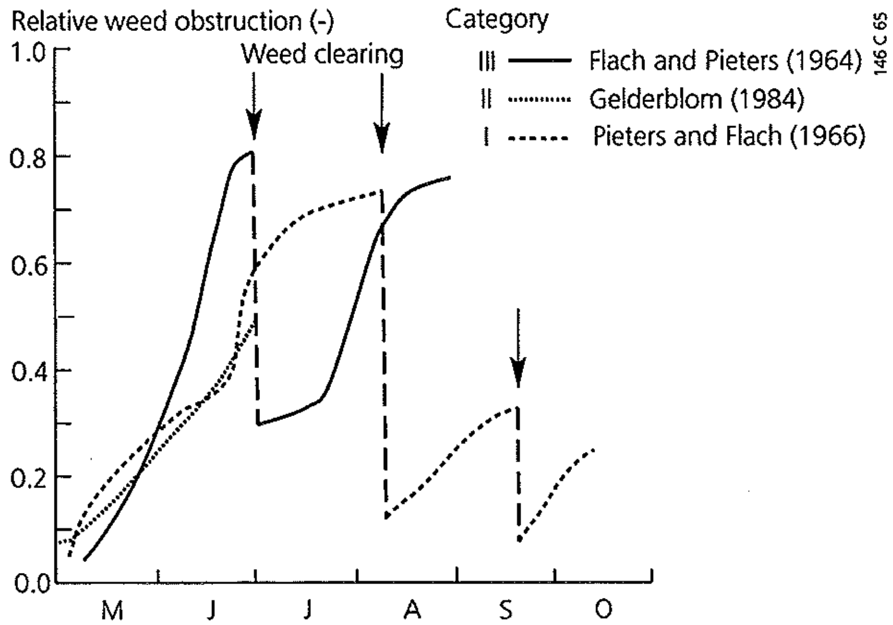


Fig. 65 Weed obstructions during the growing season obtained from field measurements in the Netherlands by different authors.

ities occur. Little information on such changes in obstructed area related to the velocity is available in the literature. Therefore I roughly estimated the change in weed obstruction from some field measurements reported in the literature and from data I had obtained in the field (see Section 7.2.1).

Two distinct situations of weed obstruction were considered (see Figure 10). When the water flows over the obstruction (Fig. 10a), its velocity has a pronounced effect on the bending of plants. Using the method discussed in Section 3.2 (KOUWEN et al., 1981), I estimated the change in obstruction brought about by the plants bending (Fig. 66). The deflection was calculated for weed lengths of 0.2 m and 0.4 m, using rigidities (resistance to bending) of 0.16 N·m^2 and 2.2 N·m^2 respectively (KOUWEN et al., 1981). In the other situation, when the weed obstruction is particularly present on the side slopes (Fig. 10b), there is very little change in obstruction. Increasing velocities result in a slightly wider effective flow area. The change in relative obstruction measured in the field (Section 7.2.1), reduces the obstruction in the order of 5% for a velocity of 0.20 m·s^{-1} and by 10% for 0.30 m·s^{-1}.

7.2 A new way to estimate the flow resistance

In the past an appropriate flow resistance value was always obtained from field measurements. For a specific flow rate, wetted area and fall in head over a certain distance, the resistance was commonly calculated by one of the flow formulae presented in Section 3.1. The net cross-section was used for the wetted area and the hydraulic radius. The result is a roughness coefficient, such as k_M, which represents

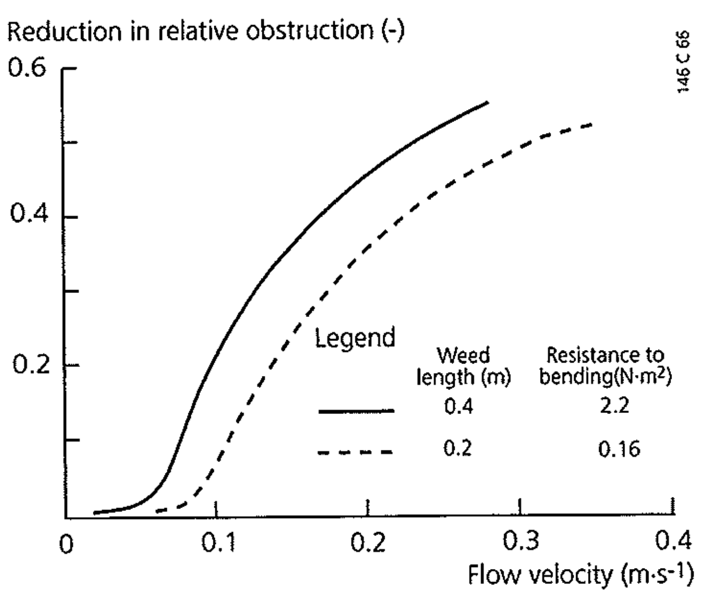


Fig. 66 *Reduction in obstruction for two weed lengths as a function of the velocity and for a water depth of 0.6 m. Increasing velocities cause the weeds to bend, which reduces the obstruction in a water course.*

mainly the roughness on the wetted perimeter. The flow formulae were originally derived for sections with little to no obstruction elements present, such as concrete lining or large alluvial channels. Severe weed growth reduces the effective flow area considerably and the flow velocity within the vegetated part is often negligible. Figure 67 shows velocities in a cross-section of a water course and is obstructed by weeds in about 50% of its cross-section. The velocity within the obstructed part is less than 10% of the velocity in the open part. Therefore the obstructed part has little or no effect on the discharge capacity of the water course. Relating the resistance to the original net cross-section will always result in different values, as shown in Figure 68. The flow area for both sections is the same, but the relative weed obstruction is different (section 1 = 0.47, section 2 = 0.68). This would lead to different k_M values if, for instance, the results shown in Figure 9 are used. The k_M value is larger for section 1 than that for section 2 (Fig. 68). These effects associated with the relationship between obstruction and roughness coefficient may partly explain the large differences in the results shown in Figure 9. Therefore it is not advisable to use such a relation between relative weed obstruction and roughness coefficient. The flow rate primarily depends on the unobstructed area, the hydraulic radius and the roughness on the edge of the unobstructed part of the water course. This is a logical way of using the flow resistance: considering only the unobstructed part and not any additional weed-covered area.

The hydraulic radius is a parameter used to express the shape of the cross-section so that different geometries can be related to a certain change in flow rate. This means it is incorrect to use the hydraulic radius (R), calculated for the total cross-section and including parts covered by weeds. Instead the hydraulic radius should be calculated for the unobstructed part only (R_o). In Table 22 the hydraulic radius is calculated for the two hypothetical sections shown in Figure 68 in both ways. The difference in hydraulic radius between the two sections is pronounced when considering the net cross-section area, but they are the same when only the unob-

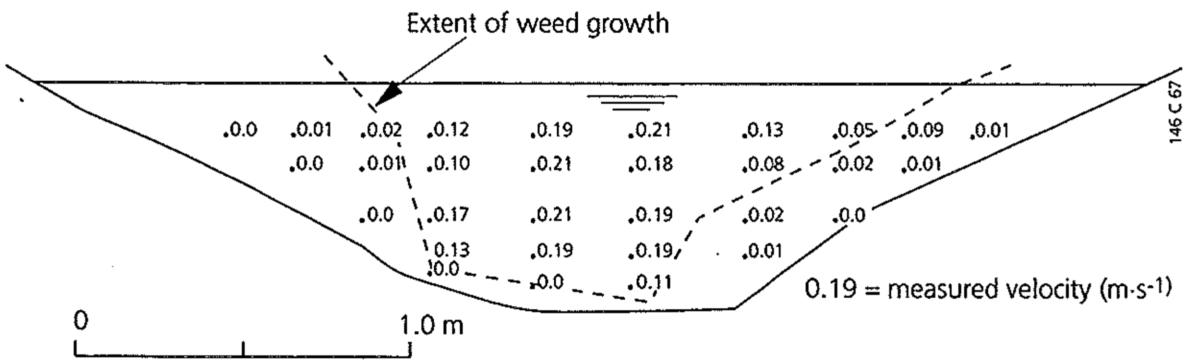


Fig. 67 Velocity measured in a cross-section of a water course where weed growth consisted mainly of Glyceria maxima (reed sweet-grass). The relative weed obstruction is about 50%.

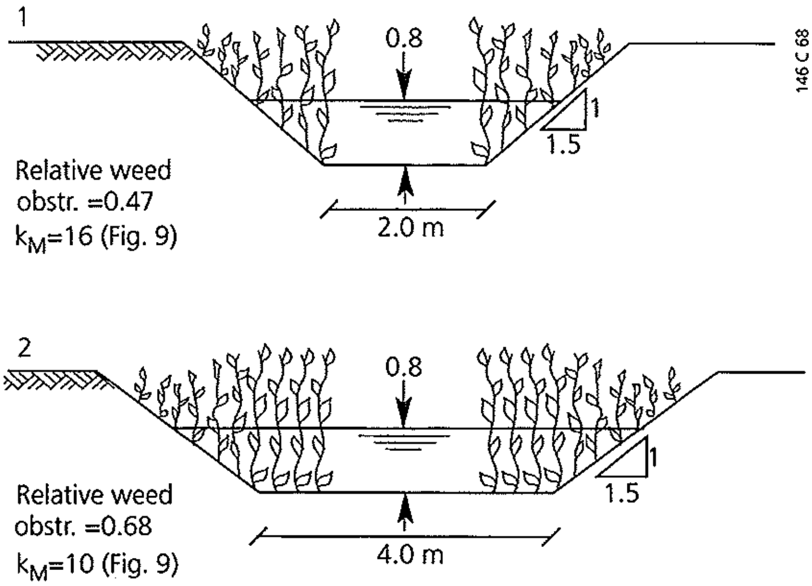


Fig. 68 *Two different hypothetical cross-sections with an equal unobstructed wetted area. The relation between relative weed obstruction and roughness coefficient (Fig. 9) gives the roughness coefficients as shown.*

structed part is considered. Using the Manning formula (Eq. 5) the flow rate depends on $R^{2/3}$. Whether the hydraulic radius is taken to be R or R_o will change the flow rate by about 18% for section 1 and 29% for section 2 (Table 22). These differences in estimated flow rate based on the two methods of using the hydraulic radius are appreciably and would also apply to the estimated roughness coefficients if the latter were estimated from field data.

Considering only the unobstructed part of the cross-section, the roughness coefficient must be calculated as:

Table 22 *Difference in flow rate (%) obtained by using the hydraulic radius for the total wetted area (R) compared with that for the unobstructed part only (R_o). The two hypothetical sections are shown in Figure 68.*

Section	Hydraulic radius				Difference in flow rate
	R (m)	R_o (m)	$R^{2/3}$	$R_o^{2/3}$	$((R^{2/3} - R_o^{2/3}) / R_o^{2/3})\cdot 100$ (%)
1	0.52	0.41	0.65	0.55	18
2	0.60	0.41	0.71	0.55	29

$$k_M^\circ = \frac{Q}{A_o R_o^{2/3} S^{1/2}} \tag{59}$$

where:

k_M° = roughness coefficient for unobstructed part ($m^{1/3} \cdot s^{-1}$)
Q = flow rate ($m^3 \cdot s^{-1}$)
A_o = area of unobstructed part (m^2)
R_o = hydraulic radius for unobstructed part (m)
S = hydraulic gradient (-)

I evaluated this proposed calculation method by doing experiments in a laboratory flume using artificial weeds. These experiments and the results are discussed in Section 7.2.1. To get some idea of the roughness coefficient for use in practice, I also used Equation (59) to estimate the roughness coefficient from some available field data. In Section 7.2.2 I calculate flow resistances by means of a numerical approach.

7.2.1 Laboratory experiments and field data

I evaluated the proposed method of using the roughness coefficient for the effective flow area only, by doing laboratory experiments carried out in a flume 8 m long. The setup of the experiment is shown in Figure 69. Weed growth was simulated in the flume by wooden pegs of 9 mm in diameter arranged in the pattern shown in Figure 69. I took measurements at five stages of weed obstruction. The water depth was within the range of 0.18-0.22 m. Each run consisted of measurements for the five weed obstruction stages as shown in Figure 69 and two flow rates (15 and 20 $l \cdot s^{-1}$). The average velocity for all the runs varied between 0.15 and 0.4 $m \cdot s^{-1}$. In addition, the bottom of the plastic lining of the flume was covered with gravel, to increase the roughness. Furthermore the roughness of the pegs was increased by gluing wood-shavings to them.

The roughness coefficient evaluated with the Manning formula (Eq. 5) gives the results shown in Figure 70a. The results showed very little difference between the two flow rates. The calculated roughness coefficients are of the same order of magnitude as field data (Fig. 9). The results of evaluating the roughness coefficient by Equation (59) are shown in Figure 70b. For a relative obstruction of less than 0.2, the roughness coefficient k_M° remains the same as the roughness coefficient k_M (see Figure 70a). If the obstruction is increased the roughness coefficients (k_M°) does not change much. For the largest relative obstruction (Fig. 70b), the flow velocity through the obstructed area becomes higher and has the effect of increasing the

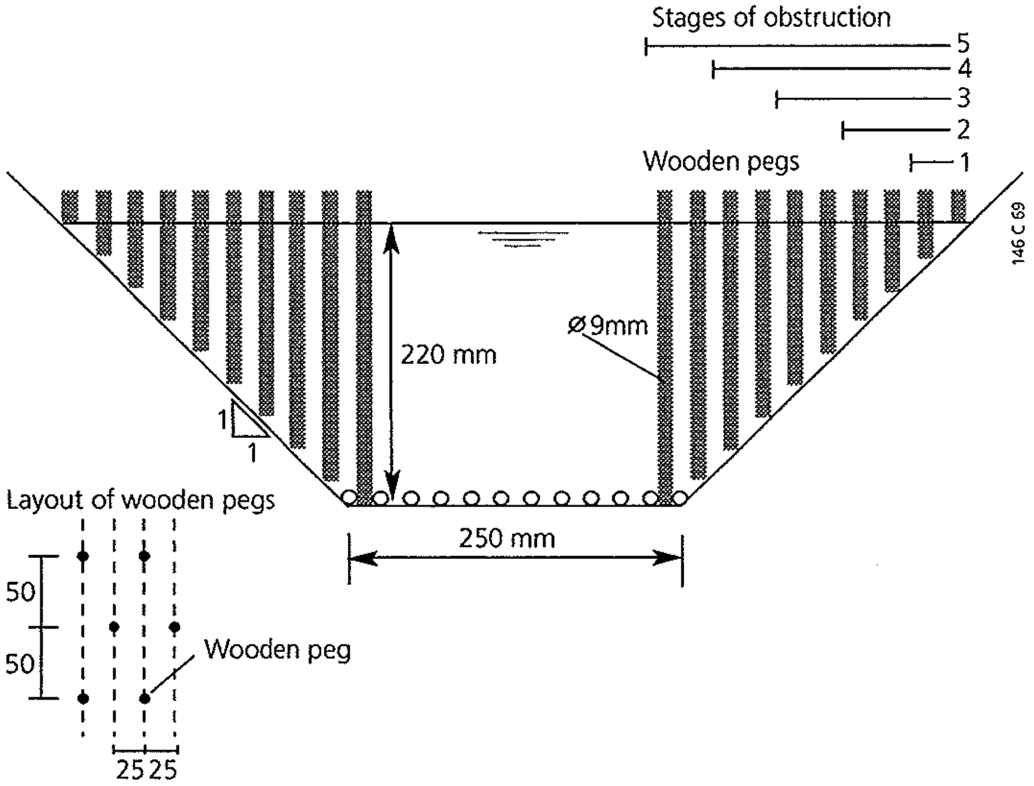


Fig. 69 Cross-section and plan view of flume, showing the artificial weeds lay-out (wooden pegs) and the 5 obstruction stages.

roughness coefficient. This increase was more pronounced for runs in which there were no wood-shavings glued to the pegs, then the pegs were smooth and caused less resistance to the flow velocity.

Table 23 shows roughness coefficients calculated from the field measurements of flow rate, hydraulic gradient and wetted area. Field data from two water courses in Salland were obtained from the Advisory Group on Vegetation Management, in Wageningen, the Netherlands (PITLO, 1990). The first water course (Zijtak) has a base width of about 0.9 m, the second water course (Soestwetering) a base width of about 1.5 m. The roughness coefficient estimated for the unobstructed area (k_M^o) gives results in the order of 30 to 34 (Table 23). Given the large variation in relative obstruction, the variation of the estimated roughness coefficient (k_M^o) is very small.

The measurements described in this section confirm expectations that the roughness coefficient would be about the same, independent of the relative weed obstruction. The method needs to be verified with more field data, before it can be applied in practice.

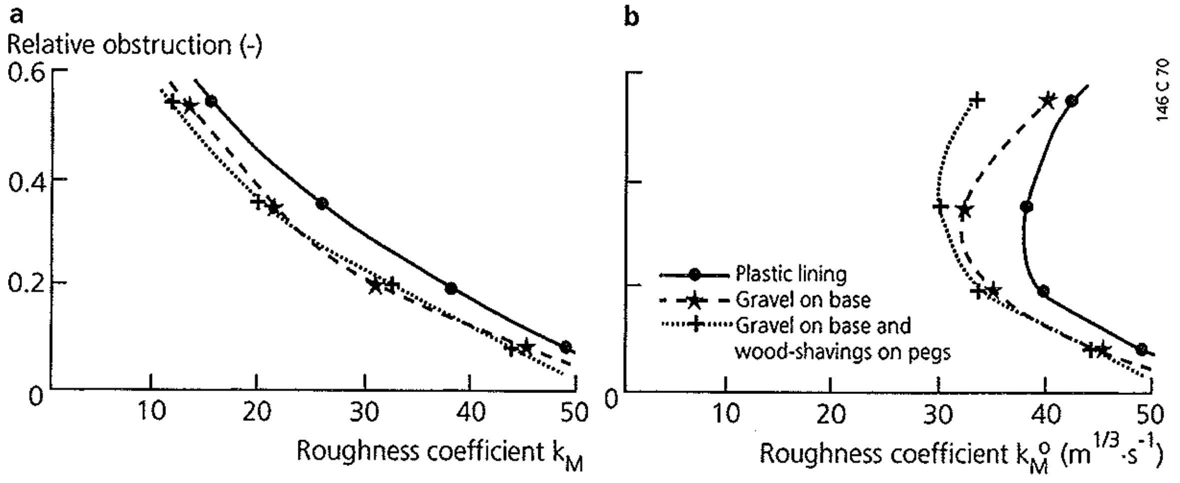


Fig. 70 *The roughness coefficient calculated for different obstructions and two flow rates (15 and 20 $l \cdot s^{-1}$), using measurements from a flume with wooden pegs (Fig. 69).*
a) Conventional method of calculating roughness coefficient k_M, using wetted area
b) Calculated roughness coefficient for the unobstructed part k_M^o (Eq. 59)

7.2.2 Calculation of velocity distribution and flow resistance by means of a numerical approach

In Section 3.4 the numerical approach used to calculate the velocity distribution in a cross-section of a water course was outlined (Fig. 12). I used this approach to estimate the flow resistance. First, the calculated velocity distribution was compared with field data. The accuracy of the predicted velocity distribution also applies to the flow rate (Q) and to the estimated roughness coefficient (see Eq. 59). As mentioned in Section 3.4.1, the numerical model can be substituted for some of the measurements of velocities in the field when estimating the flow rate. In situations without proper measuring structures the flow rate can only be obtained by measuring the flow velocity over the entire cross-section. The flow rate can be estimated from these velocities. I used the model for such situations to obtain velocities for those locations in the cross-section for which no data were available.

To evaluate the numerical approach, the model SIMVEL was applied to various channel cross-sections. Measured velocities were available from two field locations and from a laboratory flume. The field data were from the water courses given in Table 23. The laboratory set-up was as described in Section 7.2.1. I used the SIMVEL model on five situations with different relative obstructions of the wetted area. The area of a cross-section was subdivided into finite elements, as shown in Figure 71. The weed obstruction was estimated for each element from field descriptions and the measured extent of the obstructed part. The weed-obstructed area must be input into the model for use by Equation (24). Preliminary calculations yielded an obstruction of about 90% for severely vegetated parts down to zero for

Table 23 Roughness coefficient ($m^{1/3} \cdot s^{-1}$) calculated from field data by the conventional method using total wetted area (k_M using Eq. 5) and for the unobstructed area only (k_M^o using Eq. 59)

Water course	Relative obstruction (-)	Roughness coefficient k_M *	k_M^o
Zijtak	0.50	15	34
	0.80	5	30
	0.80	5	31
	0.85	3	32
Soestwetering	0.15	25	32
	0.29	26	34
	0.40	18	34

* After PITLO (1990)

the open water part. In the sensitivity analysis a range of obstruction rates was used to quantify the effect of the obstructions on the magnitude of the velocity.

Comparison of measured and calculated flow velocities

The velocity distribution calculated with the SIMVEL model was compared with measured velocities (Figure 72). Two cross-sections from the field are shown in Figure 72a. Section 1 is situated mainly in the obstructed part, for which the calculated and measured velocities compare well. In most cases, the disparity between the calculated and measured results is less than 0.03 $m \cdot s^{-1}$ for the two field locations. The laboratory experiment with the wooden pegs to simulate the weed obstruction is shown in Figure 72b. Here too, agreement between measured and calculated velocities is satisfactory. Only near the water surface does a significant disparity occur. The presence of secondary flow in the cross-section causes the actual maximum velocity to occur below the water surface (example shown in Fig. 13). The present computational method ignores this secondary flow, which results

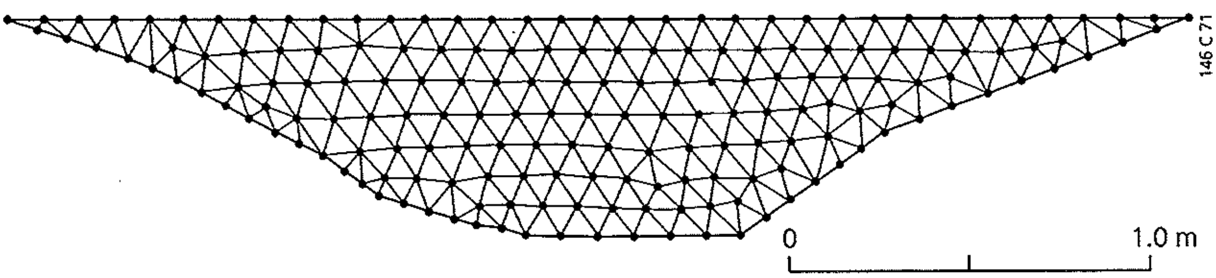


Fig. 71 Cross-section of a water course subdivided into finite elements (weed growth and velocity of this section shown in Fig. 67).

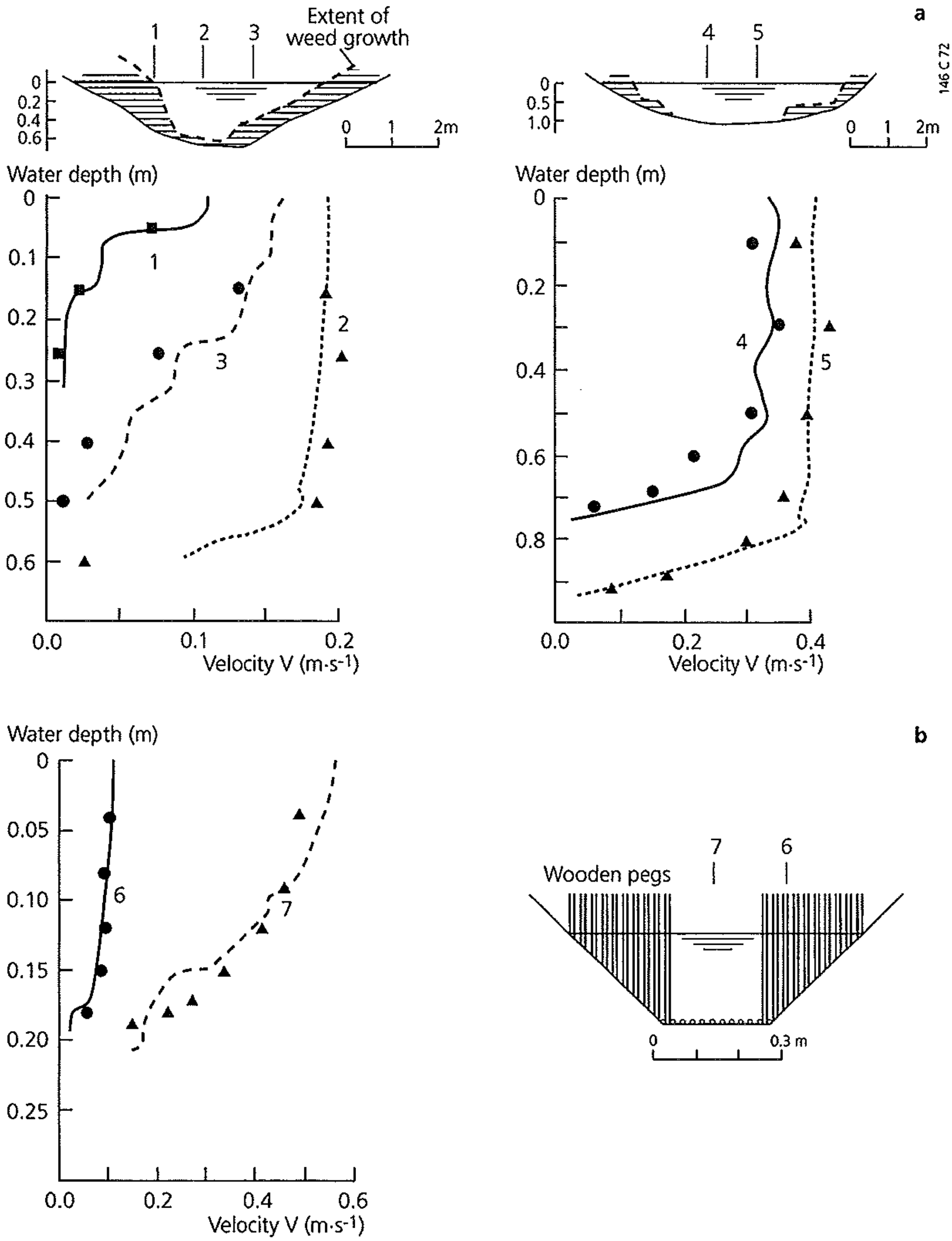


Fig. 72 *Comparison of calculated velocity profiles with the point measurements from three different cross-sections (locations of profiles are shown in the cross-sections of the water courses).*
a) Two cross-sections from water courses in Salland (Zijtak and Soestwetering)
b) Cross-section from laboratory experiment using gravel on bottom and wooden pegs with wood shavings glued to them, to simulate the weed obstruction

in the maximum velocity occurring at the water surface. This difference is particularly evident in the laboratory experiment, because of the small width to depth ratio for the open part.

Estimating the roughness coefficient

The velocity distribution calculated with the SIMVEL model also gives the flow rate. Using this flow rate I derived the roughness coefficient, k^{o}_{M}, using Equation (59) (see Table 24). The roughness coefficient applies to the open part and varies in the same order as the values obtained from applying field data to Equation (59).

Sensitivity analysis of weed obstruction

The obstruction of the severely vegetated part of the water course was varied in order to quantify its effect on the velocity. The cross-section shown in Figure 67 was used for this analysis. The estimated average velocity and the maximum velocities for a range of relative obstructions for the vegetated part are given in Table 25. Reducing the obstruction of the vegetated part from 90% down to 50% results in the flow rate increasing by 18% (from 0.118 to 0.139 $m^3 \cdot s^{-1}$).

Estimating the flow rate from numerical model and field data

The flow rate of a water course can be obtained from the model, by inputting point measurements of velocity. I varied the number of data points in order to ascertain the minimum number required. An accurate estimate of the hydraulic gradient is often not available and therefore I varied this parameter too. The cross-section used in this analysis is shown in Figure 72a (right hand side). The wetted area is about 4.4 m^2 and 50 point measurements were available. A number of measured velocities were used as prescribed values in the numerical model, as shown in Table 26. The extreme variation of the hydraulic gradient has different effects on

Table 24 The estimated roughness coefficient for the open part of the water course k^{o}_{M} obtained with the numerical model SIMVEL and compared with an estimate obtained from applying field data to Equation (59).

Water course	Relative obstruction wetted area (-)	SIMVEL ($m^{1/3} \cdot s^{-1}$)	From field data (Eq. 59) ($m^{1/3} \cdot s^{-1}$)
Zijtak	0.50	31	34
,,	0.85	33	32
,,	0.80	30	31
Soestwetering	0.40	31	34
Laboratory flume	0.56	32	32

Table 25 *Calculated flow rate, average velocity and maximum velocity for different obstructions for the vegetated part of a water course (cross-section shown in Figure 67).*

Obstruction vegetated part (%)	Flow rate ($m^3 \cdot s^{-1}$)	Average velocity ($m \cdot s^{-1}$)	Maximum velocity ($m \cdot s^{-1}$)
90	0.118	0.102	0.227
80	0.121	0.104	0.224
70	0.127	0.109	0.226
60	0.133	0.113	0.229
50	0.139	0.119	0.229

the flow rate or average velocity. The velocities remain as fixed values. The variation in velocity between these fixed points depends on the hydraulic gradient (Eq. 24) and the eddy viscosity (Eq. 31). Some distance away from the boundary the eddy viscosity has a high value, which means little shear stresses exists and the variation in velocity is minimal. Therefore using a slight gradient tends to keep the velocity the same between the fixed points and therefore the results are not very sensitive to a slight gradient, but indeed sensitive to a steeper gradient.

Conclusions

The numerical model SIMVEL is a useful tool for obtaining detailed information on the velocity distribution. The velocities calculated and measured compare well,

Table 26 *The flow rate Q ($m^3 \cdot s^{-1}$) and average velocity v_a ($m \cdot s^{-1}$) calculated with the SIMVEL model, using a number of measured velocities and different hydraulic gradients (S is the measured hydraulic gradient; cross-section shown in Figure 72a right hand side).*

Number of measurements		Hydraulic gradient			
		$S/10$	$S/5$	S	$S \cdot 5$
50	Q	1.07	1.07	1.11	1.29
	v_a	0.24	0.24	0.25	0.29
25	Q	1.00	1.01	1.07	1.31
	v_a	0.23	0.23	0.24	0.30
18	Q	0.98	0.99	1.08	1.41
	v_a	0.22	0.22	0.24	0.32
0	Q	0.35	0.49	1.09	2.45
	v_a	0.08	0.11	0.25	0.55

except for the disparity in the location of the maximum velocity. The obstruction caused by the vegetated part of the cross-section is difficult to obtain in the field, but as Table 25 shows, an accurate value is not that important. The model can be used in combination with measured data to obtain a good estimate of the velocity distribution and hence the flow rate of a water course. Even a fairly rough estimate of the hydraulic gradient is then sufficient.

7.3 Estimating the timing of weed control during the growing season

The maintenance of water courses generally focuses on controlling the weed growth (species and quantity) and on maintaining the channel dimensions. Weed control is done yearly, whereas maintenance of the channel dimensions is done once every three to ten years. Weeds are controlled either mechanically or biologically. Biological methods of weed control include: stocking with Chinese carp, which feed on the weeds; planting trees to intercept light and selective aquatic plants which remain as a thin layer on the bed and prevent other plants from growing. These biological methods are not considered in this study.
The frequency and timing of mechanical weed control depends on two main factors: the capacity of the water course and the type of equipment used. In turn these factors are governed by financial constraints. The capacity required for a water course to discharge or supply water depends on many factors, particularly on the drainage, its function, the channel's dimensions, water depth and freeboard. The possibilities and limitations of the weed-clearing equipment are governed by factors such as technical constraints, personnel and weather. In general, weed control in water courses is done between once and five times a year in the Netherlands.

Methods of deciding on weed control as described in the literature
KERN-HANSEN and HOLM (1982) described a method of aquatic weed management stipulated in Denmark. This method involves determining a maximum discharge capacity, which may not exceed a specific level. The values of the design discharge and the maximum acceptable water level are weighted in accordance with land drainage interests and nature conservation. This concept is translated into hydraulic terms by means of stage-discharge relationships. The basic discharge relation is given for a situation when the obstruction by aquatic plants is negligible. Weed control and other maintenance work can be justified, when the required discharge capacity is not sufficient.

HESS et al. (1989) evaluated river maintenance in farming areas in England in relation to its financial repercussions. The method involves estimating field drainage status from river water levels. This enables land productivity to be estimated as a function of flood levels. This creates a link between maintenance strategies and field drainage conditions. A relation has been derived in terms of the cost of weed control and its benefits for farming.

SIEFERS (1985) used empirical relations to generate discharges from rainfall data for the Netherlands. He calculated the water levels for some typical catchment areas, layout of water courses and cross-sections. He simulated the weed growth by varying the roughness coefficient k_M over the growing season. When the calculated freeboard fell below a certain threshold, weed control was done.

The three decision methods for weed control described above have quite different goals. The methods are hydraulically oriented and take drainage conditions into account. None of the methods explicitly use the growth of weeds. Therefore I developed a new approach which will be presented in Section 7.3.1. In this analysis the possibilities and limitations of the equipment for weed control are not considered. The basic components stipulating the time of weed control are therefore discharge characteristics, hydraulic criteria and weed growth. The weed growth was discussed in Section 7.1, the discharge characteristics and hydraulic capacity of a water course are discussed in Section 7.3.1. By way of an example, in Section 7.3.2 the periods when weed control is necessary in two water courses in the Poelsbeek catchment, are estimated.

7.3.1 Discharge characteristics and hydraulic capacity of a water course

The discharge from an area is in general governed by groundwater conditions encountered, such as drainage, depth of the groundwater level, seepage, land use, etc. The discharge varies over the year too, in response to the weather. In the summer period the discharge will generally be less than in the winter or early spring. On the other hand, weed growth reduces the effective flow area in summer (Fig. 73). The variation in discharge and weed growth over time depends on local conditions and control which period of the year will have the worst potential situation. The discharge characteristics, hydraulic capacity and weed growth that are important when deciding on weed control in a water course are shown in Figure 74. The maximum expected discharge must be calculated for a certain recurrence interval. For a particular water course in a catchment this maximum discharge is:

$$Q_p^t = 10\, q_p^t A_c F_c + Q_u^t \qquad (60)$$

where:

Q_p = maximum expected discharge $(m^3 \cdot s^{-1})$

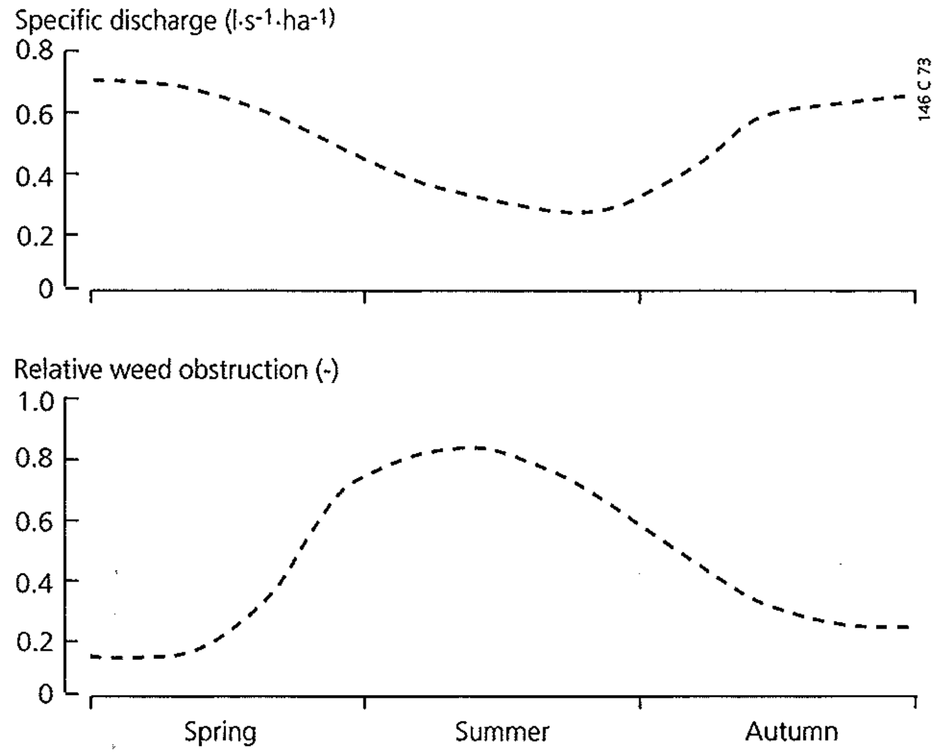


Fig. 73 *Variation in specific discharge and weed obstruction in a hypothetic water course during the annual cycle. These variations make it difficult to estimate which periods will have the worst potential situation.*

q_p = expected specific discharge for recurrence interval (factor 10 is needed to convert units from $m^3{\cdot}s^{-1}$ to $l{\cdot}s^{-1}{\cdot}ha^{-1}$)

A_c = catchment area (m^2)

F_c = reduction factor for specific discharge to account for very large catchments (-)

Q_u = additional discharges, such as effluent from sewage treatment works or sewer emergency overflow ($m^3{\cdot}s^{-1}$)

The reduction in flow area due to weed growth should be derived from field measurements such as the results shown in Figures 62 to 64. The maximum permissible capacity required for the procedure of Figure 74, can be estimated for the unobstructed part using Manning's formula as:

$$Q_m^t = A_s W_r^t k_M^o R_o^{2/3} S_m^{1/2} \tag{61}$$

where:

Q_m = maximum permissible flow rate ($m^3{\cdot}s^{-1}$)

A_s = wetted area for high flow stage (m^2)

W_r = relative weed obstruction (-)

R_o = hydraulic radius for unobstructed part (m)

S_m = maximum hydraulic gradient (-)

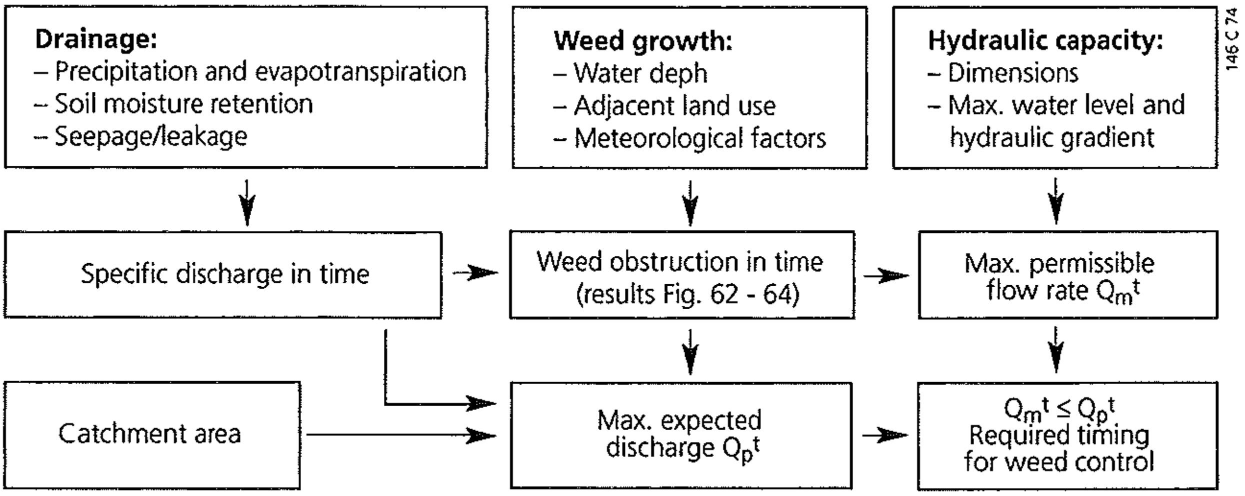


Fig. 74 Procedure to estimate periods in which weed control is required, based on information of drainage from groundwater, the weed growth (obstruction) and the hydraulic capacity of a water course.

The roughness coefficient k°_{M} must be taken in the order of 30 to 34 (Table 23). The hydraulic radius for the unobstructed part of the cross-section was derived from the field measurements of weed obstruction (Section 7.1.1). A relationship was found between the relative obstruction and the relative hydraulic radius R^1 (Fig. 75). The relative hydraulic radius is the radius for the unobstructed part divided by the radius of the net cross-section. Approximately 95% of all the results fall within the range shown in Figure 75. The hydraulic gradient required in Equation (61) will be different for each water course and can also vary during the year. The minimum freeboard required depends on the adjacent land use.

Equation (60) gives the expected maximum discharge in time and Equation (61) the maximum permissible flow rate. Weed control should be carried out when the maximum permissible flow rate, Q^{t}_{m}, is less than the expected maximum discharge Q^{t}_{p} (Fig. 74). This analysis yields the actual timing of weed control. An example of this procedure is given in Section 7.3.2. The times of weed control derived with this method are intended to be optimal for agriculture.

7.3.2 Application of procedure to estimate timing of weed control

To illustrate the procedure outlined in Figure 74 I calculated the frequency of weed control for two water courses. The maximum expected discharges for a certain recurrence interval were calculated using the unsaturated zone module of the SIMGRO model (Fig. 76). This one-dimensional model, called SIMFLOW, simulates the flow in the unsaturated zone as described in Section 4.3.4. The model uses a prescribed surface water level and a lower boundary condition to represent the

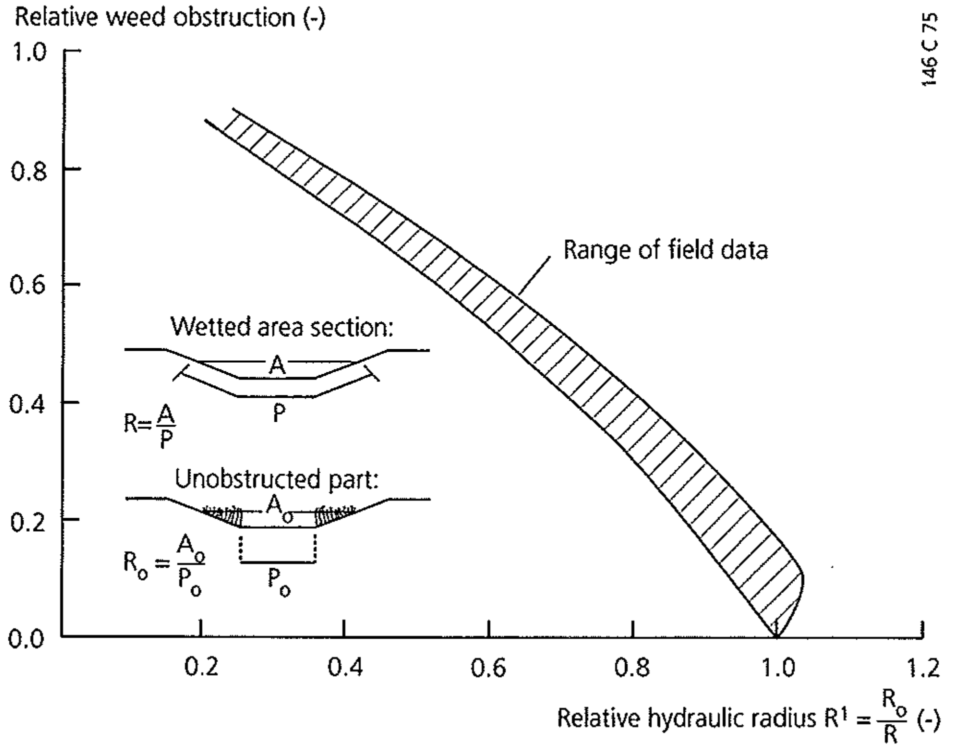


Fig. 75 *Relationship between relative hydraulic radius (R^1=R_o/R) and relative weed obstruction measured in the Poelsbeek and Bolscherbeek areas. Given a relative weed obstruction, the hydraulic radius R_o for the unobstructed part can be estimated.*

regional flow component (Fig. 76). The hydrological data for the model SIMFLOW are given in Table 27. The calculations were carried out for three typical situations defined by the 'water table fluctuation classes' commonly used in the Netherlands (Table 28). Figure 77 gives the flux for the lower boundary of the unsaturated zone model. These fluxes were calculated with a steady state groundwater model (HAAIJER, 1984) and are solely dependent on the depth of the groundwater level.

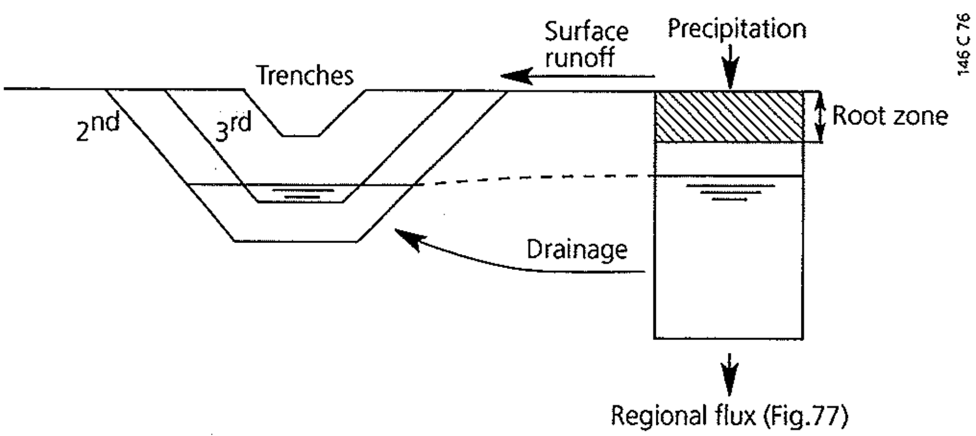


Fig. 76 *Unsaturated zone model SIMFLOW (top layer of SIMGRO model) used to calculate the drainage to the surface water system (secondary and tertiary water courses plus trenches).*

Table 27 Data for the unsaturated zone model SIMFLOW (Fig. 76) for an agricultural area (WERKGROEP WATERBEHEER NOORD-BRABANT, 1990). The hydrological situation is defined by three water table fluctuation classes described in Table 28.

Water table fluctuation class (Table 28)	Soil unit*	Depth of water course (m)		Water level below soil surface (m)**		Drainage resistance *** (d)	
		Second.	Tert.	Winter	Summer	Second.	Tert.
III	4	1.2	0.6	1.0	0.9	500	100
IV	1	1.4	0.8	1.2	1.0	750	200
VII	2	-	1.2	-	-	-	3000

* For description of soil units see Table 16
** Water can be supplied in summer to guarantee this level
*** Drainage resistance of trenches = 75 d

Table 28 The seasonal fluctuation of Dutch groundwater levels as a depth below soil surface is defined in terms of 'average highest' and 'average lowest' groundwater level (VAN DER SLUIS and DE GRUIJTER, 1985).

Groundwater level	Water table fluctuation class		
	III	IV	VII
'Average highest'	< 0.4	> 0.4	0.8-1.4
'Average lowest'	0.8-1.2	0.8-1.2	> 1.2

For leakage conditions this is sufficiently accurate, but for seepage the flux may change during and over the years. These relations must be regarded as simplifications of the fluxes which may be calculated with regional models like SIMGRO or MOGROW. A surface runoff of 2% was assumed for rainfall between 10 and 20 $mm \cdot d^{-1}$ and 3% for rainfall events of more than 20 $mm \cdot d^{-1}$ (Table 9). These percentages of surface runoff were also used in the simulations of the Hupsel catchment and gave good results (Section 5.4). See Table 15 for the land use taken in these calculations.

Discharges

The hydrological calculations were carried out for a 40 year period (1951–1990), using daily meteorological data from the Royal Dutch Meteorological Office (weather station De Bilt) and a time step of one day. The specific discharge obtained during the year is shown in Figure 78. The discharge with a recurrence interval of

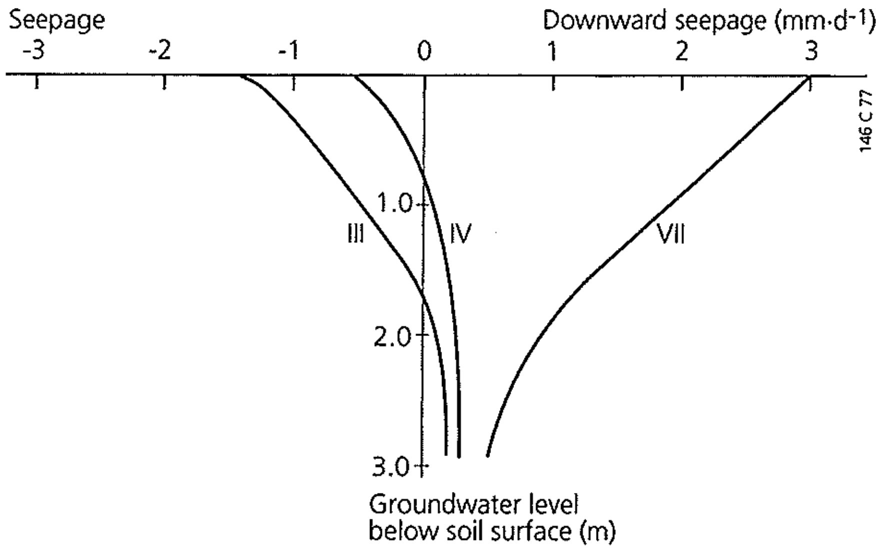


Fig. 77 *Lower boundary flux used in the model SIMFLOW for three water table fluctuation classes defined in Table 28 (HAAIJER, 1984).*

once in a year is slightly lower than the discharge considered when designing new water courses. For water table class III the calculated maximum discharge in December is about 0.95 $l{\cdot}s^{-1}{\cdot}ha^{-1}$ (Fig. 78a). For this class the recommended discharge, for designing water courses in the Netherlands, is 1.2 $l{\cdot}s^{-1}{\cdot}ha^{-1}$ (CULTUUR-

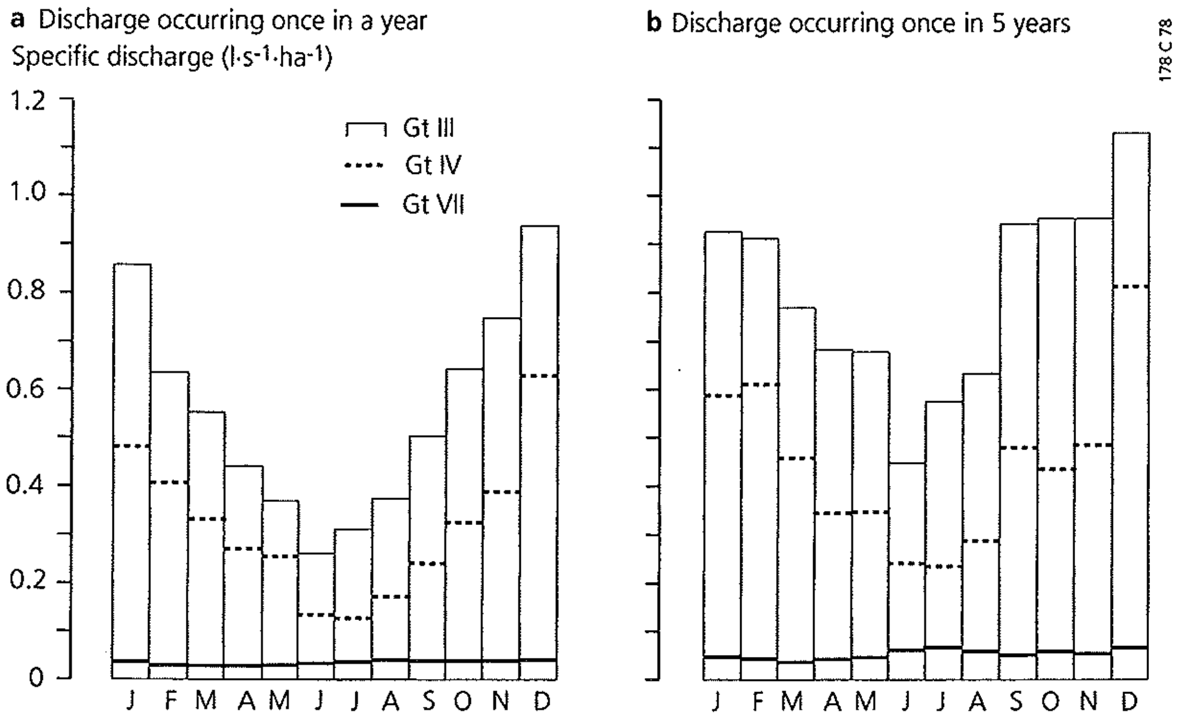


Fig. 78 *Specific discharge calculated with the model SIMFLOW for the three water table fluctuation classes (definition given in Table 28), using the data from Table 27 (met. data from 1951 to 1990).*

TECHNISCH VADEMECUM, 1988). For water table classes IV and VII the calculated maximum discharges are 0.65 and 0.05 $l \cdot s^{-1} \cdot ha^{-1}$ respectively. A sudden decrease in discharge can be noted between these two classes. The design discharges are 1.0 $l \cdot s^{-1} \cdot ha^{-1}$ for class IV and 0.33 $l \cdot s^{-1} \cdot ha^{-1}$ for class VII (CULTUUR-TECHNISCH VADEMECUM, 1988). For the classes III and IV the specific discharge in summer is much less than in winter. In summer this reduction can be used to tolerate a fair amount of weed growth in the water courses.

The frequency of exceedance was also plotted on semi-logarithmic paper (Figure 79), together with the generally applied relation as reported by BLAAUW (1962). The simulated results show a nearly straight line, but the return period of once in 5 years (0.2 times per year in Fig. 79a) occurs at a lower discharge than would be expected from the relation given by Blaauw. For the summer period (May to August) the simulations yield a line that is far from straight, as illustrated by the results for the month of June shown in Figure 79b. In the summer period the retention characteristics of the groundwater system have a pronounced effect on

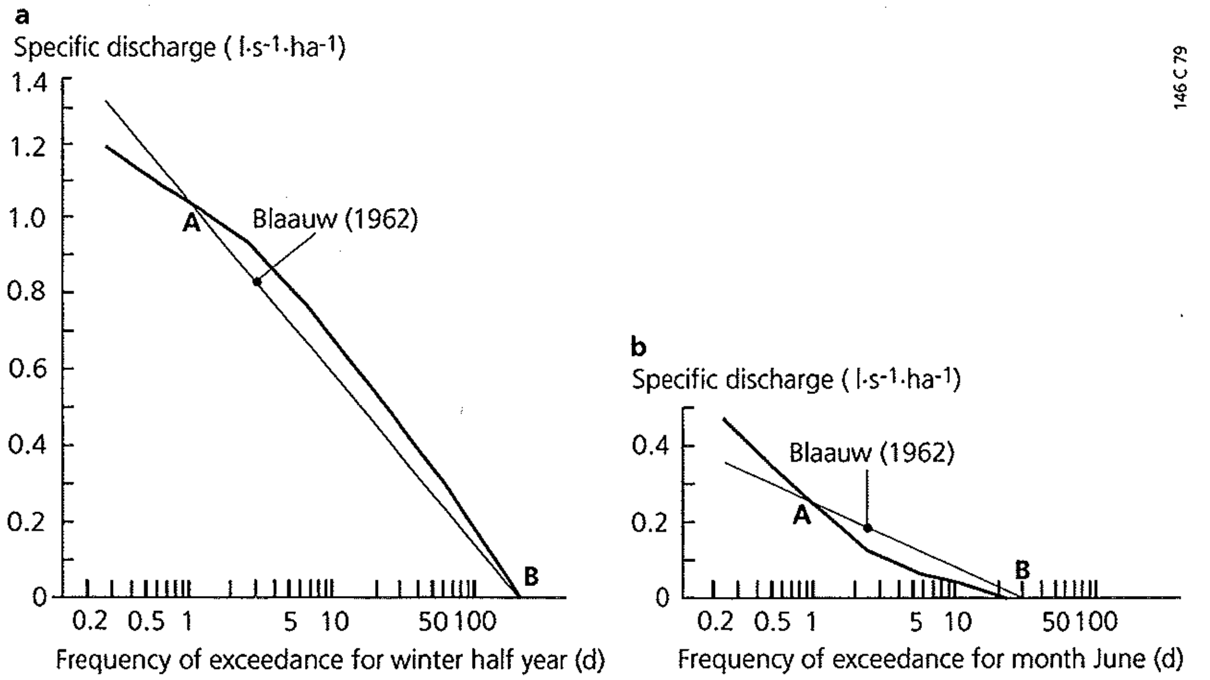


Fig. 79 Specific discharge for groundwater fluctuation class III, given as a frequency of exceedance (solid line). The relationship from Blaauw (1962) is shown by a straight line through the points with a discharge having an exceedance of once a year (point A) and the zero discharge (point B; exceedance of 182 times for winter half year and 30 times for June).
a) Winter half year
b) June

the discharge. But for major rainfall events, occurring less frequently, the retention becomes less important and the discharge is larger. The variation of the groundwater depth for the simulations is shown in Figure 80, using the daily calculated groundwater levels. Point A in Figure 80 is a groundwater level which is on the average exceeded 14 days a year. Point B is a groundwater level which is on the average 14 days a year lower.

Timing of weed control

The specific discharge with a recurrence interval of 5 years (Fig. 78b) was chosen to indicate the weed control required. For the summer period this recurrence interval was considered to be appropriate in relation to the economic risk. Two water courses in which the weed growth had been measured, were selected for the analysis. They were locations 2 and 4 in the Poelsbeek catchment (for location see Figure 60, for hydraulic data see Table 29). The hydraulic radius for the open water part was obtained from the relation shown in Figure 75. The roughness coefficient for the open water part, k°_{M}, was taken as 32 (Fig. 70b). Before the first weed clearance the relative obstructions shown in Figure 62 were taken, thereafter the increase in relative obstruction were taken (for category I shown in Figure 64). The reduction in relative obstruction brought about by high velocities was assumed to begin at a velocity of 0.10 $m{\cdot}s^{-1}$ and reaches a maximum reduction of 10% for 0.30 $m{\cdot}s^{-1}$ (see Section 7.1.2).

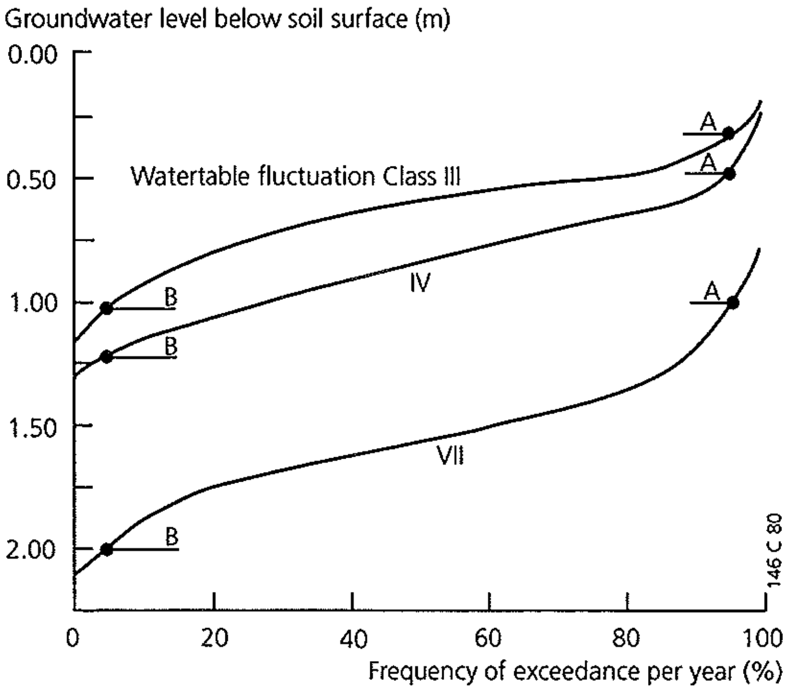


Fig. 80 Duration curves for daily calculated groundwater levels by the model SIMFLOW. The three water table fluctuation classes are defined in Table 28 (for points A and B see text).

Table 29 Hydraulic design data for two water courses in the Poelsbeek area (locations 2 and 4 shown on Figure 60)

Parameter	Location	
	2	4
Maximum hydraulic gradient ($m.km^{-1}$)	0.85	0.35
Base width (m)	1.4	2.2
Wetted area for high flow stage (m^2)	2.0	4.4
Design discharge ($l \cdot s^{-1} \cdot ha^{-1}$)	1.2	1.2
Weed category*	Ia	IIIm
Catchment area (ha)	866	1603

* a = average obstruction of the observed range (see Fig. 62-64)
m = maximum obstruction of the observed range

The calculations were first carried out for the three hydrological conditions given in Table 27 separately. Table 30 shows the weed control required during the growing season. As well as the clearings required during the growing season, one clearing is required before the winter. Weed control is deemed to be required when the maximum expected discharge (Q_p^t) exceeds the maximum permissible flow rate (Q_m^t). This means when the ratio Q_p^t/Q_m^t, also shown in Table 30, would exceed unity. A low ratio of Q_p^t/Q_m^t indicates a great safety factor against too high water levels. If the entire area had been water table fluctuation class III, the frequency of weed control would have been three times a year (end of June, mid August and before winter - see Table 30). For class IV the frequency would be twice a year and for class VII only once a year. The necessary clearing frequency is low, which is influenced by the specific discharge of 1.2 $l \cdot s^{-1} \cdot ha^{-1}$ used for the design of the two water courses. The need for weed control is more pronounced in August than in other months. This month is characterized by an increasing specific discharge (Fig. 78) and maximum weed obstruction.

The weed control required was also calculated for the actual groundwater fluctuation classes present in the catchment area. The sewer emergency excess flow from Haaksbergen (Fig. 60) was also included. The maximum excess flow for design purpose is 30 mm of rainfall from the impermeable areas, lasting 75 minutes (WATERSCHAP REGGE EN DINKEL, 1992). This excess flow is generally assumed not to coincide with the maximum drainage. Instead an amount of 10 $mm \cdot d^{-1}$ (1.16 $l \cdot s^{-1} \cdot ha^{-1}$) was assumed to occur at the same time as the peak discharges from drainage. At location 2 the areas of the three water fluctuation classes and the surfaced part from Haaksbergen are: Gt III = 44%; Gt IV = 32%; Gt VII = 17% and surfaced areas = 7%. At location 4 the areas are: Gt III = 47%; Gt IV = 36%; Gt VII

Table 30 Required timing of weed control for two locations in the Poelsbeek, when the ratio Q_p^t/Q_m^t (maximum expected discharge divided by maximum permissible flow rate) would exceed unity (expected discharges occur once in 5 years; xxxx = estimated clearing period during growing season; ==== = clearing period before winter).

Loca-tion	Groundw. Class	May		June		July		Aug.		Sept.	Oct.
2	III				xxx			xxx		=========	
			0.60		-		0.80		0.81		
	IV							xxx		=========	
			0.34		0.44		0.59		0.50		
	VII									=========	
			0.07		0.12		0.14		0.14		
4	III			xxx			xxx			=========	
			0.61		0.55		0.88		0.83		
	IV					xxx				=========	
			0.37		0.69		0.27		0.51		
	VII									=========	
			0.06		0.19		0.32		0.22		

= 13% and surfaced areas = 4%. The frequency of weed clearance required for both locations is twice a year, as given in Table 31. The first clearing in summer is slightly later than required for the water table class III given in Table 30. Location 4 requires earlier clearance, because the maximum hydraulic gradient is less than at location 2.

The frequency is dependent on the assumptions used in the groundwater model. If, for instance, more surface runoff occurs than assumed at present, this would increase the specific discharge and also the required frequency of weed control. As pointed out in Section 7.1.1 the weed growth characteristics are based on measured obstructions in two growing seasons. More measurements are needed to improve the reliability of these obstruction curves.

Conclusion

The procedure outlined in this Section can be used to determine the frequency of weed control. As shown in Table 30, different water table fluctuation classes each require a particular period of weed control. The proposed calculation procedure

Table 31 *Required timing of weed control for two locations in the Poelsbeek, when the ratio Q_p^t/Q_m^t (maximum expected discharge divided by maximum permissible flow rate) would exceed unity. Catchment is assumed to consist of the groundwater fluctuation classes III, IV and VII, together with urban area (discharges occur once in 5 years; xxxx = estimated clearing period during growing season; ==== = clearing period before winter).*

Location	Periods when clearance is required												
		May		June		July		Aug.		Sept.		Oct.	
2							xxx				==========		
			0.49		0.76		-		0.65				
4					xxx						==========		
			0.53		-		0.52		0.85				

enables the clearing periods to be determined for a number of locations in a catchment. This information can be used by water boards to adjust their weed control programme so that equipment is used as efficiently as possible.

8 Evaluation of weed control strategies for the Poelsbeek and Bolscherbeek area

I evaluated weed control strategies on the basis of both a cost-benefit analysis and a multi-objective decision analysis. The evaluation procedures were described in Chapter 2 and shown in Figure 2. The description of the Poelsbeek and Bolscherbeek pilot areas plus the hydrological calculations with the MOGROW model were given in Chapter 6. Below the alternative weed control strategies are evaluated on the basis of costs and benefits (i.e. to minimize possible financial loss). This approach gives the maintenance costs for the water board and the expected financial losses for agriculture, without considering the environmental impact on weed control. In Section 8.1 the weed control alternatives to be analysed are described, together with the weed growth and aspects of clearing that are included in the MOGROW model. In Section 8.2 the hydrological information required to be able to estimate the financial loss is described and Section 8.3 gives the cost-benefit analysis. I also applied a multi-objective decision analysis which included environmental aspects. The methodology and required input data, together with the results are given in Section 8.4. The conclusions drawn from this case study are summarized in Section 8.5.

8.1 Weed control options

In the pilot area the present frequency of weed control is generally three times a year. Some of the larger water courses which are deeper and therefore take longer to become overgrown, are cleared only twice a year. If a cold spring is followed by a dry early summer, this frequency is applied to more water courses. In my analysis I assumed weed control was three times a year: twice during the growing season and once before winter. This schedule is henceforth referred to as the 'reference situation'. The alternative schedules are based on the results obtained in Section 7.3.2 and on discussions with staff from some water boards, and are summarized in Table 32, together with the periods of weed clearance. Because weed control is costly and is ecologically disruptive, the options formulated in Table 32 all involve less frequent weed removal. Options 2 and 3 have a frequency of twice a year but differ in the timing of the first clearing. Clearing the bottom once during the growing season (option 4, Table 32) assumes reducing the relative weed obstruction by 35%, as compared with the obstruction before clearing. Clearing one side slope and the bottom once during the growing season, assumes reducing the obstruction by 65% (option 5, Table 32). These figures have been derived from the field data (Section 7.1.1). The option of clearing both side slopes only, is ecologically unsatisfac-

*Table 32 Periods of weed control for the seven options formulated (*** = weed removal over entire cross-section; bb = bottom only and sb = one side slope and bottom).*

Option	May	Jun	Jul	Aug	Sep	Oct	Explanation
1	****	****			********		Reference run
2	****				********		1st delay ½ month
3		****			********		1 month delay
4	bbbb				********		Bottom only
5		sbsb			********		1 side slope, bottom
6					******		Before winter only
7				******			In August only

tory and was therefore ignored. Options 6 and 7 have only one clearing, but the clearing in August (option 7) requires in addition the removal of some weeds locally before winter. This incidental clearing was not taken into account in the cost analysis.

The measurements from the pilot area presented in Section 7.1.1 were used to characterize the weed growth in the water courses. Before the first weed clearance it was assumed that the data on obstruction shown in Figure 62 is applied, thereafter the increase in relative obstruction was assumed (for category I see Figure 64b). The data shown in Figs. 62 and 64 show a range in relative obstructions for the three classes, but only the maximum values for each class in time have been assumed for the simulations. Considering this extreme situation for the weed obstruction I assumed it to be on the save side. On the other hand calculated peak discharges tend to be a bit lower than reality (Figs. 41 and 55); so these two aspects will cancel each other out.

The MOGROW model takes into account the weed clearance by specifying the starting time, the sequence of water courses to clear and the speed at which the equipment clears the weed. This speed depends on the stage of weed growth and the type of clearing. It ranges between 3 and 4 $km \cdot d^{-1}$ for options 1 to 5, and is 2 $km \cdot d^{-1}$ for options 6 and 7. In accordance with the real situation, the weed clearing starts in the Bolscherbeek followed by the Poelsbeek. The water courses in the catchment are always cleared in the upstream direction. The clearing of a particular water course is skipped when the relative obstruction is less than 30%. During the growing season this percentage reflects the common practice (option 1) of skipping one clearing event in the larger water courses.

8.2 Hydrological aspects

The simulations with the MOGROW model require an appreciable amount of computer time (Section 6.3). Therefore the analyses were performed for a limited number of meteorological years only. To select these years on the basis of peak discharges calculated during summer I used the groundwater flow model SIMFLOW. The hydrological simulations carried out in Section 7.3.2 with this one-dimensional groundwater flow model over a 40-year period yielded daily discharges. These discharges were analysed to identify any extreme wet conditions per month of the growing season. The hydrological conditions represented by a groundwater fluctuation class III were selected for this analysis (see Table 27). For weed control a number of peak discharges occurring in a month were assumed to give critical situations. Therefore the four maximum discharges in each month were used to obtain an average peak discharge. The decision to use four maximum discharges per month is arbitrary, but it is more realistic than using only one value. The average peak discharges for each month over the 40 years were then used for each of these months, to estimate the recurrence interval. Table 33 gives the selected years together with the estimated recurrence interval of the discharge peaks. For instance, once in about 3 years the peak discharges for the month May are equal or exceed the peaks occurring in May 1985. In 1985 the entire summer was rather wet (recurrence interval for discharge in each month is 3 to 5 years). The other years selected had wet conditions in early, mid or late summer (Table 33). The discharge conditions for 1981, 1980 and 1988 are referred to as moderate wet conditions and the other years (1979, 1966 and 1954) as extreme wet conditions. Table 33 only gives the recurrence intervals for the months with higher peak discharges than the median conditions (recurrence interval of 2 years or more).

Table 33 Recurrence interval for peak discharges calculated with the model SIMFLOW over the period 1951-1990 obtained by taking the four highest peaks occurring each month.

Year	Month					Discharge conditions	
	May	June	July	Aug.	Sept.	Period	Characterization
1985	3	5	3	3	3	Entire	Moderate
1981	6	4	7			Early	,,
1980		3	5	2		Middle	,,
1988			4	3	6	End	,,
1979	20	6	2			Early	Extreme
1966		20	13	3	3	Middle	,,
1954		3	10	6	10	End	,,

The simulations with the MOGROW model were carried out for the seven years specified in Table 33 and for the seven weed control options (Table 32), using the hydrological data described in Section 6.2. The hydrological parameters important for the evaluation of weed control strategies are high water levels (flooding), high stream flow velocities, high groundwater levels and changes in evapotranspiration. In each simulation these data were recorded over the period important for agriculture (early May to mid September). I defined a high water level as a channel water level that is less than 0.3 m below the top of the bank, and a high groundwater level as a level less than 0.3 m below the ground surface. All the channel water levels and groundwater levels exceeding this criterion were recorded, so that the expected financial loss could be estimated (Section 8.3.2). The flow velocities exceeding 0.50 $m \cdot s^{-1}$ were also recorded. The starting date of the calculations was taken well before the beginning of summer, to avoid influences from assumed initial conditions, such as groundwater levels and moisture storage in the root zone. Therefore each simulation covered the period January to October. To obtain information about the different hydrological conditions within the catchments, the output data were differentiated per catchment (Fig. 51). These extreme hydrological conditions were used for both the cost analysis (Section 8.3) and the multi-objective decision analysis (Section 8.4).

The impact of two weed control options, calculated with the MOGROW model for the surface water levels and groundwater levels of node 185, is shown in Figure 81 (for location of node 185, see Figure 52). The levels shown are for options 1 and 6 of Table 32, using meteorological data of 1954. That year was characterized by extreme wet conditions from mid July onwards (Table 33), resulting in higher surface water levels and also higher groundwater levels (Fig. 81). For option 6, the weed clearing at the end of summer only, results in a sudden fall of the surface water level, but the groundwater level changes much slower (Fig. 81). The results shown for node 185 are one of the extreme differences in calculated levels; most nodes show considerably smaller differences. As indicated by Figure 81 different weed control strategies may result in higher groundwater levels over the period July until November. The extremely wet conditions result in higher groundwater levels, but changes in water table fluctuation class (for definition see Table 28) are unlikely.

The evapotranspiration over the summer period did not differ much between the seven weed control options (no more than 2 mm for each of the years considered: see Table 33). The evapotranspiration calculated by the MOGROW model for wet conditions (waterlogging) was not reduced (Fig. 30, line I). I assumed no reduction in evapotranspiration under wet conditions, as it affects only the transpiration of a crop. Therefore changes in evapotranspiration could not be used directly to account

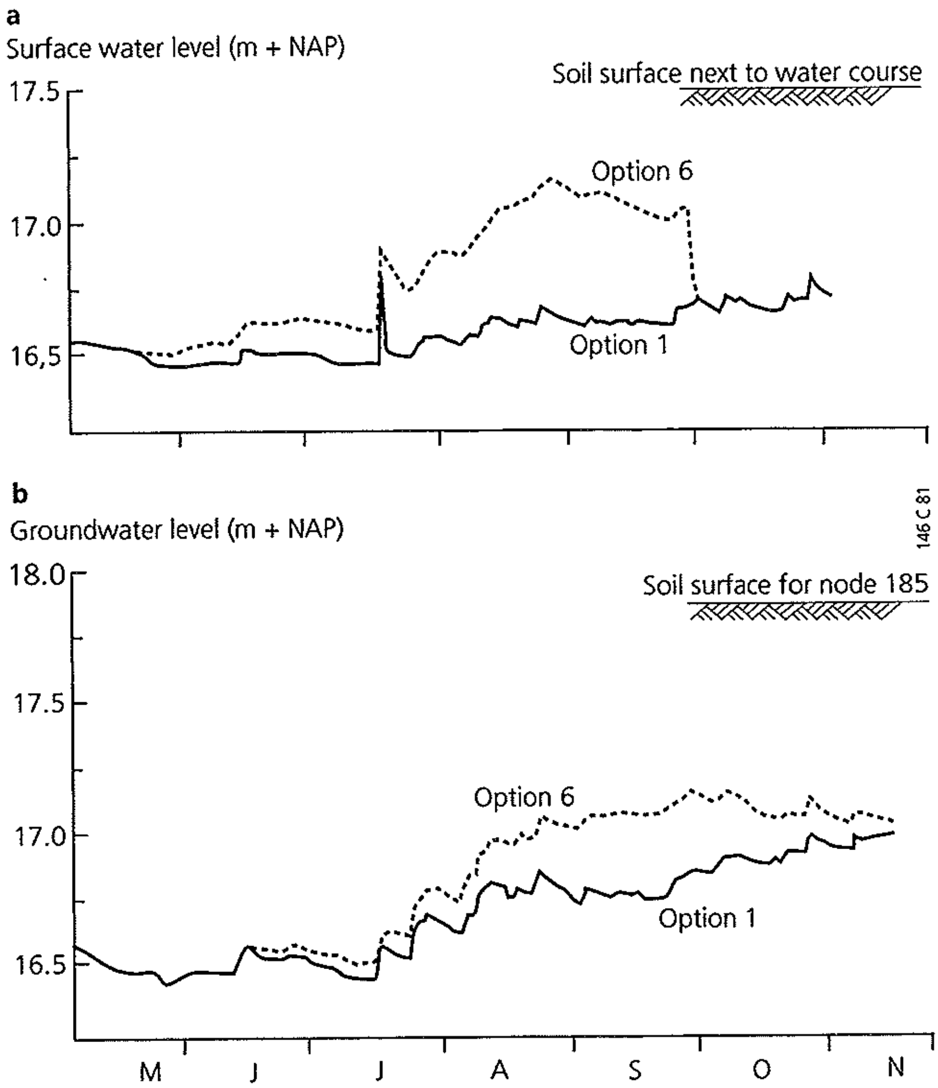


Fig. 81 *Difference in surface and groundwater levels between options 1 and 6 of Table 32, calculated with the MOGROW model for node 185, using meteorological data for 1954 (location of node 185 shown in Fig. 51).*
a) Surface water levels
b) Groundwater levels

for the reduction in crop production.

The MOGROW model schematizes the groundwater system by a number of nodes spaced about 500 m apart (Figure 52). The water courses are modelled by nodes some 250–1000 m apart. This means that very localized situations cannot be revealed by the MOGROW model and therefore do not influence the results of the evaluation procedure.

8.3 Cost-benefit analysis

The weed control strategies were evaluated by comparing the costs of weed control against the financial damage to agriculture that would probably be incurred if the water courses are cleared less frequently. The residential areas in the catchment are located on the higher ground and therefore will not be affected. The price-level used in this section are considered for a common reference year of 1990.

8.3.1 Cost of weed control

As discussed in Section 2.3 some data on the cost of weed control were available. Unfortunately they were not sufficiently detailed and could not be used in this analysis. Table 1 indicated the cost of weed control (single event), together with the range in cost incurred by the different water boards. These results were used to estimate the costs involved, using expert judgement (see Table 34).

8.3.2 Financial loss to agriculture caused by the extreme hydrological conditions

The hydrological conditions causing financial losses are high water levels in the water courses, high stream flow velocities and high groundwater levels (Section 8.2). The difficulty of assessing the financial loss to agriculture was outlined in Section 2.3. To overcome this difficulty I made certain assumptions.

Surface water levels

Near the water course the soil surface is usually somewhat lower than the average soil surface. Along the water course there are also variations in the ground surface. These lower-lying area will be inundated if the water in the water course overtops the banks (Fig. 82). I therefore attributed these local damage to fields and inconvenience to the farmer's management to high water levels in the water courses.

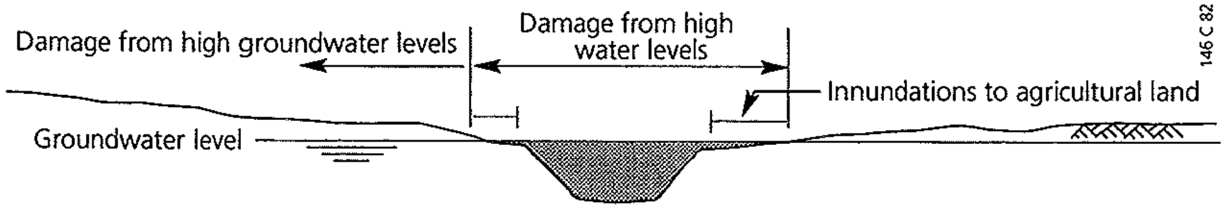


Fig. 82 Section through water course showing the local inundations of agricultural land next to the water course. This financial loss is included in the 'damage functions' for the surface water levels.

Table 34 Cost of weed control (NLG·m^{-1}) for the seven weed control options of Table 32, for various widths of water course (SIEFERS, 1992; WATERSCHAP REGGE EN DINKEL, 1992).

Bottom width (m)	Weed control options				
	1	2/3	4	5	6/7
0 - 1	2.10	1.50	1.25	1.40	0.85
1 - 2	2.25	1.60	1.30	1.45	0.90
2 - 4	2.85	2.00	1.55	1.80	1.10
> 4	3.15	2.20	1.75	1.90	1.20

I used expert judgement to arrive at the estimates of damage given in Table 35. The density of the major water courses of the catchment and modelled in MOGROW is about 10 m·ha^{-1}. This can be regarded as a theoretical spacing between water courses of 1000 m (class 4 and 5 of Table 17). The smaller water courses (branches) not considered in the surface water model will also cause flooding. Assumed is that 20% of the land between the major water courses is flooded, which would mean per 100 m length of water course considered in MOGROW an inundated area 200 m wide, being 2 ha. A dairy farm has approximately a grass production of 10 t·ha^{-1} dry matter, having a value of 3000 NLG·ha^{-1}. Based on a growing season of 120 days this comes down to 25 NLG·ha^{-1}·d^{-1} (VAN BAKEL, 1993). A flooding of this land for one day gives a damage of 50 NLG. For inconvenience of farming management the total damage is assumed double being 100 NLG for a water course of 100 m in length (Table 35). This damage is used for water levels less than 0.1 m below the adjacent ground surface. For water levels between 0.1 and 0.2 m below ground surface the area inundated was assumed to be 2%, giving a damage of 10 NLG for 100 m length of water course (Table 35).

Stream flow velocities

High velocities in the water courses will cause erosion of the sandy bed and also damage the protecting side slopes. These effects will be more pronounced in the

Table 35 Financial loss due to high surface water levels resulting in local inundation or flooding of pasture as shown in Figure 82. Damage includes the inconveniencing of farming management and applies to one day of high water levels.

Surface water level below soil surface (m)	Damage per 100 m of water course (NLG)
< 0.1	100
0.1 - 0.2	10
0.2 - 0.3	1

bends of a water course. The average threshold velocity at which erosion begins in the sandy soils of the pilot area is about 0.4-0.5 $m \cdot s^{-1}$ (CHOW, 1959). Using this value I estimated the probable erosion as a percentage of the water course length (Table 36), using expert judgement. I assumed the cost of repair per unit length of water course is 100 $NLG \cdot m^{-1}$, based on prices given by ANDRÉ et al. (1991).

Groundwater levels

Different weed control strategies can result in groundwater levels being too high during several days. I considered the effect of these short-lived high groundwater levels on maize to be negligible. Furthermore maize is generally grown on the higher-lying plots. However, the financial loss resulting from damage to pasture can be significant as will be outlined below.

I based the damage function for pasture on the results reported by PEERBOOM (1990) and POSTMA (1992). For a dairy farm they computed the reduction in grass yield resulting from high groundwater levels and from farming practices such as: traffic, cattle grazing, stock relocating, etc. All these losses were estimated for various meteorological years and expressed as a percentage of the net grass yield. The loss related to the occurrence of high groundwater levels is given by the sum of daily exceedances of a typical reference groundwater level, as:

$$SOS = \sum_{i=1}^{N} 100\ (h_i - h_r) \qquad \text{for } h_i > h_r \tag{62}$$

where:

SOS = sum of daily exceedances of a typical reference groundwater level (cm·d)
h_i = groundwater level (m)
h_r = reference groundwater level (m)
N = number of days the groundwater level exceeds the reference groundwater level (-)

For the sandy soils present in the pilot area (Table 16) I assumed this reference level was 0.30 m below soil surface (BEUVING, 1992; POSTMA, 1992). For ground-

Tabel 36 *Length of water course affected by erosion due to high stream flow velocities; the damage applies to one day of such velocities.*

Velocity ($m \cdot s^{-1}$)	Channel length damaged by erosion (% of length)
0.5 - 0.6	1
0.6 - 0.7	2
> 0.7	5

water levels higher than 0.2 m below soil surface, the grass cover would suffer serious damage from the trampling cattle and therefore the cattle will be moved indoors (PEERBOOM, 1990). In Figure 83 the results are shown for the sandy soils present in the pilot area. The estimated range in damage presented in Figure 83 was based on simulations for the periods June to September in the years 1965-1986 (POSTMA, 1992). For the cost analysis I took a damage of 10% at *SOS*=400 cm·d (Fig. 83). A dairy farm with a total grass production of about 10 t·ha^{-1} dry matter, would suffer this loss in production of 10% or 1 t·ha^{-1}, which has a value of about 300 NLG·ha^{-1} (based on a selling price for grass of 0.30 NLG.kg^{-1} dry matter).

8.3.3 Results of cost-benefit analysis

For the Poelsbeek and Bolscherbeek the water board Regge en Dinkel receives revenues from the farmers and citizen on total about 150 NLG·ha^{-1}. The expenditure for weed control by the water board is about 44 NLG·ha^{-1} (WATERSCHAP REGGE EN DINKEL, 1992).

Poelsbeek

The financial loss caused by high surface water levels, high stream flow velocities and high groundwater levels is presented for the Poelsbeek catchment in Table 37, together with the cost of weed control. The meteorological conditions causing a variation in discharge characteristics (Table 33) have a distinct effect on the results. In the Poelsbeek area the damage is primarily caused by high groundwater

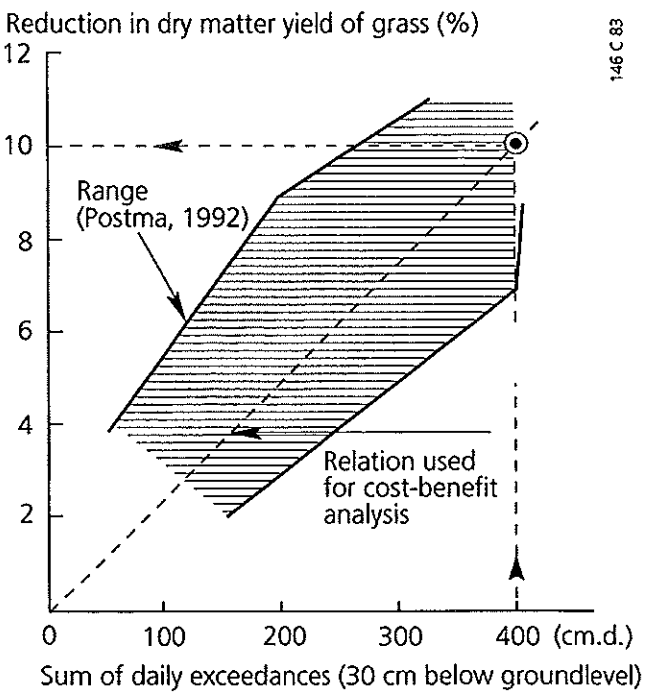


Fig. 83 *Range in damage to the dry matter yield of grass (expressed as percentage reduction of potential yield) due to high groundwater levels and from farming practices, over the period June to September, for sandy soils (adapted from POSTMA, 1992).*

levels in combination with a few high surface water levels. In most years a weed control frequency of twice a year or even once a year causes a small increase in 'damage', compared to option 1. The meteorological years with moderate peak discharges (1985, 1981, 1980 and 1988), result in smaller financial losses (Table 37). Even only one clearing (options 6 and 7) at the end of summer does not increase the losses much. The years with extreme peak discharges (1979, 1966 and 1954) result in greater losses. The wet conditions occurring in 1966 during the middle of summer, in combination with maximum obstructions in the water courses, results in the worst possible conditions. Even with three weed controls in such a year, the financial losses are much greater than for 1979 and 1954, which also have extreme peak discharges (Table 37). All the simulations assume the surface water management remains unchanged, but in reality under the extremely wet conditions of 1966 the water boards would have lowered the weirs to a level that is common during winter and would have cleared weeds in some water courses which are most prone to overflowing. This would result in reality in much smaller financial losses than the simulations for 1966 predicted (Table 37).

Except for 1966, the differences in financial losses for the options 2, 3, 4 and 5 are very small. Therefore any one of these options can be chosen. Options 6 and 7 show a large increase in financial losses for 1966 and a much smaller increase in the losses for the years 1980 and 1954. Therefore these options still remain feasible solutions.

Table 37 Predicted financial loss (NLG·1000) in Poelsbeek catchment for the summer season caused by the combined effect of high surface water levels, high stream flow velocities and high groundwater levels. The costs of weed control are also presented. Weed control options are described in Table 32.

Years	Characterization of discharge	Weed control options						
		1	2	3	4	5	6	7
1985	Moderate	11	10	10	10	10	9	9
1981	,,	12	12	12	12	12	11	11
1980	,,	18	18	17	18	17	24	24
1988	,,	8	9	8	9	8	7	7
1979	Extreme	26	26	27	24	27	26	26
1966	,,	154	188	169	187	168	694	657
1954	,,	19	19	19	19	19	30	28
Cost of weed control		184	131	131	94	108	74	74

From a hydrological point of view, a weed control frequency of twice a year is permissible. A saving on weed control of NLG 53,000 per year (equal to 12 NLG·ha^{-1} of total area or 19 NLG·ha^{-1} for the agricultural area) could be achieved in the Poelsbeek area (Table 37), by reducing the weed control frequency from three times to twice a year. Only for 1966 is the difference in financial loss between these frequencies of weed control substantial: in the order of NLG 34,000 (difference between options 1 and 2 of Table 37). The annual saving on weed control costs makes it possible to cover the losses occurring in the occasional wet year, such as 1966. The recurrence interval of such wet conditions is once in about 13 to 20 years (see Table 33). The change from twice a year to even once a year is also feasible, but this does of course increase the risk of financial loss.

Bolscherbeek

The financial losses caused by high surface water levels, high stream flow velocities and high groundwater levels in the Bolscherbeek catchment is presented in Table 38, together with the cost of weed control. In the Bolscherbeek area high stream flow velocities are the primary factor for the financial loss. The estimated losses for the seven simulation years behave very similarly to those in Poelsbeek, but they are much smaller. The saving in the cost of weed control achieved by reducing the frequency from three times a year to twice a year is, per unit area, about the same as in Poelsbeek (Table 38).

Table 38 Predicted financial loss (NLG·1000) in Bolscherbeek catchment for the summer season caused by the combined effect of high surface water levels, high stream flow velocities and high groundwater levels. The costs of weed control are also presented. Weed control options are described in Table 32.

Years	Characterization of discharge	Weed control options						
		1	2	3	4	5	6	7
1985	Moderate	0	1	0	1	0	6	6
1981	,,	0	3	2	2	2	6	6
1980	,,	0	1	1	1	0	8	8
1988	,,	0	0	0	0	0	3	3
1979	Extreme	18	18	34	18	34	35	35
1966	,,	110	143	126	143	125	241	225
1954	,,	1	6	5	6	5	23	18
Cost of weed control		103	74	74	53	62	41	41

Sensitivity analysis

A sensitivity analysis of the 'damage functions' for surface water levels, stream flow velocities and groundwater levels was carried out. The prices used for the damage functions were multiplied by a factor 2, one at a time. For the Poelsbeek area the losses are nearly twice the amount when the damage function for groundwater levels is double. For the Bolscherbeek area the losses are about twice the amount for the stream flow velocities, except for options 6 and 7. For those options the losses are caused by the high water levels. Even with these increased losses, the conclusions drawn from the results given in Tables 37 and 38 remain unchanged. In that case the weed control frequency of twice a year remains feasible, but the financial losses resulting from clearing only once a year become rather large.

The losses were differentiated for the upper and lower parts of the Poelsbeek catchment (see Fig. 51). The losses in the upper part are slightly greater than for the lower part, but the variation in the losses was similar to that presented in Table 37. The increased risk for some locations in the upper part of the catchment has also been reported by the water board. The upper catchment has a flat portion with shallower groundwater levels and a larger upward seepage flux.

8.4 Multi-objective decision analysis using the ELECTRE II method

This integrated water management approach for weed control also incorporates the ecological impact. In Chapter 2 I outlined the approach to follow when using the ELECTRE II method (GOICOECHEA et al., 1982) to analyse weed control strategies. The hydrological parameters described in Section 8.2 were also used in the multi-objective decision analysis. In Section 8.4.1 the ELECTRE II method is further discussed. A description of the formulated objectives and goals follows (Section 8.4.2). In Section 8.4.3 the results of the evaluation are presented.

8.4.1 The theory underlying the ELECTRE II method

I outlined the multi-objective decision analysis in Section 2.4. The ELECTRE II method was selected because it enables the evaluation of discrete options and uses quantitative values to represent the criteria. Each criterion is also assigned a weight to express its relative importance. The procedure is focused on the ranking of alternatives. Often there is no dominant solution, and therefore a set of comparable options has to be analysed, using a minimum level of acceptability and a maximum level of allowable discordance. In ELECTRE II the preference is calculated using a concord index $C_e(i,j)$ as:

$$C_e(\mathrm{i,j}) = \frac{W^+ + 0.5W^=}{W^+ + W^= + W^-} \tag{63}$$

where:
W^+ = sum of weights for criteria where i is preferred j
$W^=$ = sum of weights for criteria where i and j are equivalent
W^- = sum of weights for criteria where j is preferred to i

The concord index expresses the number of criteria for which option i is preferred to option j (i>j), using the sum of weights for these criteria. The discord index is a measure of the maximum deviation obtained by adopting the options pertaining to a criterion. This enables the discord to be expressed and only those options that yield results close to the maximum value to be selected. The discord index D_e(i,j) is defined as:

$$D_e(\mathrm{i,j}) = \frac{\text{maximum difference where i<j}}{\text{range of scale } S_k} \tag{64}$$

The range of scale, S_k, can be defined per criterion to control the maximum disparity in the results of two options. A large value of S_k would mean that large disparities can be accepted.

In the ELECTRE II method two extreme outranking relationships are defined: a strong and a weak relationship. The strong relationship R_s is defined by the conditions:

$$\left.\begin{array}{llcll} C_e(\mathrm{i,j}) & > p^* & \text{or} & C_e(\mathrm{i,j}) & > p^\circ \\ D_e(\mathrm{i,j}) & < q^* & & D_e(\mathrm{i,j}) & < q^\circ \\ W^+ & > W^- & & W^+ & > W^- \end{array}\right\} \tag{65}$$

If at least one set of the above conditions is satisfied, then option i strongly outranks option j. The weak relationship R_w is defined by the conditions:

$$\left.\begin{array}{ll} C_e(\mathrm{i,j}) & > p^- \\ D_e(\mathrm{i,j}) & < q^* \\ W^+ & > W^- \end{array}\right\} \tag{66}$$

where:
p^* = weight level of strong concordance
p° = weight level of average concordance

p^- = weight level of weak concordance
q^* = weight level for strong discordance space
q^o = weight level for weak discordance space

The strong and weak relationships (Eq. 65 and 66) are used to rank the options, given a set of weight levels for concordance and discordance. These data are used to obtain the preferability between options. The selection or ranking of the options can be presented graphically. An arc is drawn from option i to option j to depict the preference relationship between alternatives (Fig. 84). The graphs are used in an iterative procedure to obtain the final ranking (see GOICOECHEA et al., 1982).

8.4.2 Goals for weed control

The purpose of a decision analysis is to find which option meets the objectives stipulated by the decision maker. For my analysis of weed control options the important goals (see Fig. 1 too) were:

Hydrological:
- Drainage conditions must be suitable for local agriculture
  Insufficient drainage will cause waterlogging. The general effects of such wet conditions include: impeded access of machinery; reduced transpiration (=reduced crop production); damage to soil and grass cover by trampling of cows; etc.
- Flood protection
  Protection against flooding of agricultural land, roads and residential areas
- Scour protection
  Too high stream flow velocities will cause erosion and damage to the bank protection (grass cover).

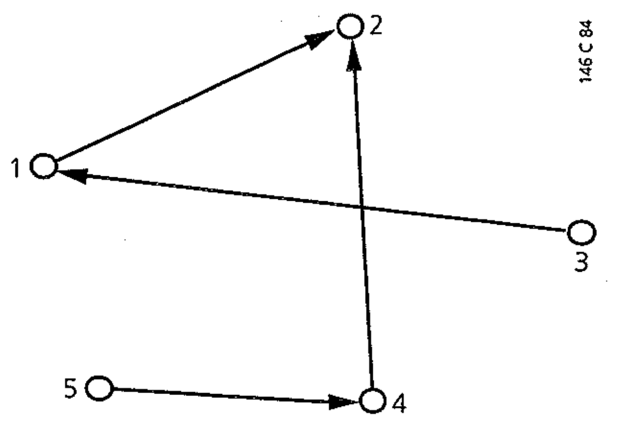


Fig. 84 The preference relationships between options are shown by arcs (e.g. option 4 is preferred above option 2).

Other:
- Minimize cost of weed control
  More frequent weed control means greater costs for the water board. Less frequent weed control could result in the removal of more weeds, which might require different equipment.
- Reduce adverse ecological impact
  Aim to carry out weed control in a period in which damage to the terrestrial and aquatic vegetation is minimal.

In the Netherlands, many of the designated 'nature areas' require wet conditions, but excessive changes in water levels may be harmful to certain plant species too. In my analysis I did not investigate the impact of different weed control strategies on woodland and nature areas.

The goals given above were translated into specific criteria. In Table 39 the payoff matrix is presented for each option to each criterion. A numerical value is used for each option and each criterion to express how 'good' or 'bad' this option is. The hydrological criteria can be expressed quantitatively. The simulations carried out with the MOGROW model give the number of high surface water levels, high stream flow velocities and high groundwater levels (Section 8.2). The financial losses calculated using these extreme conditions in the cost-benefit analysis were input in ELECTRE II (results of Tables 37 and 38). The cost of weed control for both pilot areas was also given in these Tables 37 and 38. For this criterion a lower value is preferred (saving on cost of weed control), therefore the costs were multiplied by −1, before being input in ELECTRE II (Table 39). I arrived at the other values after discussions with a number of water boards and scientists from research institutes. These values are to some extent arbitrary. To maintain a proper grass cover which protects the side slopes from erosion, it is very important to clear weeds regularly during the growing season. The values given in Table 39 are therefore based on the number of clearings carried out on the side slopes during the growing season. The ecological impact given in Table 39 has been split up into the impact on aquatic and terrestrial vegetation. A maximum rating of 5 has been considered for this criterion and all the other criteria in Table 39. Preferably, the aquatic vegetation should be cleared at the end of the growing season, in order to preserve some of the rare species (TER STEGE and POT, 1991). For the terrestrial vegetation, clearing in the middle of summer is preferable. Ecologists prefer clearing part of the section instead of the entire section (Table 39, options 4 and 5), because the aquatic and terrestrial vegetation in a water course is used by birds, amphibians and other fauna as a refuge. The clearing of the entire section reduces the biological diversity of aquatic vegetation and increases the likelihood of rare species disappearing. The type of equipment used depends on how much weed has

Table 39 The payoff matrix for the weed control options of Table 32 that were used for the ELECTRE II analysis (for references and explanation of values see text). For the hydrological conditions the damages given in Table 37 (Poelsbeek) and Table 38 (Bolscherbeek) were used.

Criteria		Weed control options						
		1	2	3	4	5	6	7
Weed control:								
. Cost	Poelsbeek	-184	-131	-131	-94	-108	-74	-74
	Bolscherbeek	-103	-74	-74	-53	-62	-41	-41
. Damage to grass cover		2	1	1	0	0.5	0	0.5
Ecological impact:								
. Aquatic		0	1	1.5	1	1.5	5	4
. Terrestrial		0	1	3	2	5	0	0.5
Equipment:								
. Availability		2	1.5	1.5	1	1	0	0
. Light machinery		5	5	4	2	2.5	0	0.5

to be cleared. The equipment currently used is not very suitable for clearing the large amounts of weeds, which would be encountered if only one clearing was done per year (options 6 or 7). This aspect is reflected in the criterion of availability of the equipment (Table 39). Also, the equipment suitable for less frequent clearance will be larger and heavier. The use of heavier equipment could result in more damage to the maintenance road next to the water course, and this equipment is also more cumbersome and may not be able to gain access to all sites.

8.4.3 Results of ELECTRE II

The weights assumed for the 3 runs presented in Table 40 reflect the importance attached to the criteria. The present strategy of the water board is represented by run a1 which considers agriculture to be paramount and the ecological impact less important. Run a2 is focused in total on agriculture benefits, whereas in run a3 the ecological aspects are paramount and agriculture is less important.

The results of the analysis with the ELECTRE II method for the three strategies are summarized in Table 41. The most promising weed control options (ranked 1st and 2nd) are given for the seven simulation years of Table 33. The weight levels adopted for the concordance and discordance space (Eq. 65 and 66) are also given in Table 41. Runs a1 and a2 reflect a preference for agriculture. For the present strat-

Table 40 Weights to express the importance for the criteria, resulting in three typical weed control strategies.

Criteria	Strategies		
	Present situation (run a1)	Agriculture (run a2)	Nature conserv. (run a3)
Hydrological conditions:			
. high water levels, velocities and groundwater levels	10	20	10
Weed control:			
. Cost	10	20	10
. Damage to grass cover	5	10	5
Ecological impact:			
. Aquatic	5	5	20
. Terrestrial	5	5	20
Equipment:			
. Availability	10	10	10
. Light machinery	5	5	5

egy the 3rd option is ranked first for all the seven simulation years for the Poelsbeek and the Bolscherbeek. Various options are ranked second, but the 2nd and 5th options appear very frequently. The strategy focusing on agriculture gives various options as the best. The moderate discharge conditions of 1985 gives the 7th weed control option as the best solution for the Poelsbeek area, because of the saving on maintenance cost. For the other moderate years the 6th weed control option is preferred, and for the extreme years the 4th option. Options 1, 3 and 5 are very favourable for the Bolscherbeek. For the extreme discharges conditions in the Bolscherbeek, the best option turns out to be option 1 (The only time it is ranked first: Table 41).

For nature conservation (run a3 in Table 41) weed control options 5 and 3 seem to be the best choice. Options 6 or 7 are not often selected as favourite options, because the availability of the required equipment is a constraint.

The weight levels for the concordance and discordance space used for the results presented in Table 41 were varied in order to assess the effect of variations on the ranking. The values were increased and decreased by 0.1, one at a time. Only the increase of the concordance condition resulted in slightly different rankings for run a2. These small differences do not change the conclusions drawn above.

Table 41 The weed control options ranked 1^{st} and 2^{nd} for Poelsbeek and Bolscherbeek obtained for the strategies and discharge conditions described in Table 33.

Run	Strategy	Ranking	Discharge conditions Moderate		Extreme	Moderate		Extreme
			1985	80/81/88	54/66/79	1985	80/81/88	54/66/79
			Poelsbeek			Bolscherbeek		
a1	Present	1	3	3	3	3	3	3
	situation	2	7	2,5,6	2,4,5	1,2,5	2,5	1,2,5
a2	Agricult.	1	7	6	4	3,5	5	1
		2	6	3,5,7	1,5	1	1,3	5
a3	Nature	1	5	5	5	5	5	5
	conserv.	2	3,7	3	3	3	3	3

Note: Concordance and discordance conditions for Eqs. 65 and 66 are: $p^- = 0.60$, $p^\circ = 0.65$, $p^* = 0.75$, $q^\circ = 0.20$, $q^* = 0.65$

8.5 Conclusions about weed control strategies

I evaluated the weed control alternatives by the cost-benefit analysis and the multi-objective decision analysis. These two methods used different objectives and therefore the results cannot be compared.

In the cost-benefit analysis one single objective was considered: minimizing financial loss to agriculture. A weed control frequency of two or three times a year (see options 1 to 5 from Table 32), gives very little difference in damage. A weed control of once a year increases the financial loss, but this could be easily offset by the considerable saving in the cost of weed control. Weed control once a year is also feasible in this catchment, because the expected discharges are lower than used for the design of the water courses. The dimensions have been based on a specific discharge of 1.2 $l{\cdot}s^{-1}{\cdot}ha^{-1}$, but the expected discharges calculated with a one-dimensional groundwater model are in the order of 0.6–1.0 $l{\cdot}s^{-1}{\cdot}ha^{-1}$ (see Fig. 78).

The multi-objective decision method ELECTRE II has the advantage of taking into account a wide range of objectives affected by the weed control. The options preferred depends to some extent on how the preference is formulated. These preferences are part of the appraisals given to the criteria (Table 39) and of the importance to be assigned to each criterion (Table 40). The ranking of alternatives obtained as a result of ELECTRE II does not give a quantative answer about the difference between the options. For instance, the options ranked first and second may be nearly as good, but the third one may be much less preferred. The strategy referred to as 'present situation' prefers the option with weed control carried out twice a year. For the Poelsbeek area the options with only one clearing before winter are preferred when agriculture is paramount (run a2; option 6 and 7 of Table 41), particular for the years with moderate discharge conditions. This result is based on economic considerations, in which less frequent weed control is preferable (cheaper). For the Bolscherbeek area the options 1, 3 and 5 are more feasible. For the ecological impact (run a3 of Table 41) the 5th option is preferred.

The information obtained by the cost-benefit analysis and the multi-objective decision method can be used by the water boards to modify their weed control programmes so that equipment is used as efficiently as possible. The best time of weed control can be determined in accordance with the local conditions, such as discharge, channel dimensions and function of a water course (agriculture, nature areas, etc.).

9 Conclusions and recommendations

9.1 Conclusions

The research presented in this thesis focused on the weed control carried out by the water boards. A combined surface and groundwater flow model was developed for this practical water management problem. The model was verified in the Hupselse Beek area and applied to the Poelsbeek and Bolscherbeek area to obtain operational rules for weed control. The following conclusions can be drawn for the related topics considered in this research.

Combined surface and groundwater flow model MOGROW

This simulation model is a simplified representation of the complex hydrological system. These simplifications of reality impose restrictions on the use of a model. The input data required for the model are always more than what are readily available. The simplifications and the accuracy of the input data are very important, when evaluating the output.

In such a combined surface and groundwater flow model the calculated groundwater levels or the discharges never match the observed values exactly. In turn there is always a temptation to increase the detail of the schematization in order to improve the results. This again makes the model more complicated and less user-friendly. Also a more detailed schematization requires additional input data. A simulation model of this size and nature requires its users to have a considerable hydrological knowledge (important for schematization) and experience (important for collecting input data and analysing the results).

There was insufficient field data available to enable me to do a complete calibration and verification of the MOGROW model in the Hupsel and in the Poelsbeek/Bolscherbeek area. It proved fairly simple to simulate groundwater levels in both catchments reasonable. For most of the observation wells, fluctuations in the calculated levels over the year were the same as the observed fluctuations. The mean standard deviation for these situations is less than 0.20 m, a value which indicates that the calculated groundwater levels are reliable (Tables 6 and 20). The measured discharges are more difficult for the model to reproduce. One of the shortcomings is that simulated peak discharges are less than observed (Figs. 45 and 55). The simulations in Hupsel reveal that preferential flow and surface runoff cannot be ignored in hydrological models. Preferential flow results in less retention in the unsaturated zone and this results in higher groundwater levels and higher peak discharges. Figure 49 shows a proposed schematization of the unsaturated zone, in which the preferential flow path is considered; each land use is divided up into a column representing the preferential flow and a column for the stagnant soil moisture. For water quality problems this process must be included in the modelling.

Reasonable agreement was obtained between the calculated and measured hydrographs of the Bolscherbeek catchment (Fig. 56). This catchment receives effluent from two sewage treatment works, and occasionally an emergency overflow from the sewer system of the town of Haaksbergen. This rather complex situation was simplified considerably in the simulation model (Fig. 31).

The standard deviation is a poor way of comparing calculated and measured hydrographs. It gives one value for a wide range of discharges. Therefore the calculated and measured hydrographs should be given as a frequency of exceedance per year (Fig. 57). When the discharge is presented in this way, the differences between calculated and measured discharges are apparent. Further refinement is possible by making the comparison per season.

The MOGROW model has been kept fairly simple in order to handle practical applications in which simulations over several years are required. The application to Hupsel required 30 minutes of CPU time to simulate one year on a VAX 4200. For the Poelsbeek and Bolscherbeek area 3 hours of CPU time were necessary, because the surface water schematization (number of water courses and weirs is 212) in that area is about three times larger. Such a schematization is still feasible, but more complex situations, particularly with respect to the surface water, will require inordinate amounts of computational time and input data. Also, the output of such an exercise will be excessive and difficult to analyse properly.

The use of the hydrological model focused on the application to evaluate weed control strategies. But a physically based hydrological model like MOGROW could be applied to all kinds of water management problems in the Netherlands and abroad.

Flow resistance

In the past the roughness coefficient k_M has been derived from field measurements, using the total wetted area of the cross-section. Less attention has been paid to the weed growth stage and often different values for the flow resistance have been derived, which could not be explained (Fig. 9). In my research I estimated the roughness coefficient for the unobstructed part of the cross-section only. Measurements of velocities in two water courses showed that the flow rate through the obstructed part is generally very small and can be ignored when calculating the roughness coefficient. The roughness coefficient k_M^{o} for the unobstructed part derived from field data and laboratory experiments was found to vary between 30 and 34 $m^{1/3} \cdot s^{-1}$.

I also took the hydraulic radius for the unobstructed part only. The advantage of using the roughness coefficient in this manner is that it can be used in combination with data on relative weed obstruction in the surface water model (Eq. 61).

Using a numerical approach to estimate velocity distribution

The SIMVEL model simulates the velocity distribution in a cross-section of a water course, using the finite element method. I adapted the model to take account of weed growth. The difference between calculated and measured velocities was found to be quite small. The calculated velocity distribution was also used to derive a roughness coefficient for the unobstructed part. The roughness coefficient derived in this manner did not differ much from those estimated from the field measurements directly.

The numerical model was also used in combination with field measurements. This enables the flow rate to be estimated from point measurements of velocity. The model SIMVEL calculates the velocity distribution using a set of field measurements as boundary conditions. Experiments with only 18 measurements, instead of the 50 measurements taken originally, still gave a good estimate of the average velocity or the flow rate. For the model an accurate measurement of the hydraulic gradient is not necessary, as long as the value used is not larger than in reality (Table 26).

Weed obstruction

The phenomenon of weed growth is very complicated, the dynamics of growth are to some extent unpredictable. You never know exactly when and where a weed species will burgeon and in what quantity. Therefore weed growth processes cannot be modelled easily and is difficult to integrate in a hydrological model. A practical solution was found, using the relative weed obstruction in the water courses to represent the weed growth. The measurements were grouped into three classes according to the average water depth (Figs. 62 and 63). At some locations nutrient losses from the adjacent land use, particularly heavily fertilized maize fields, stimulates excessive weed growth (stinging nettle). No large variations in relative obstructions were obtained between the growing seasons of 1990 and 1991.

Weed control

To date the water boards determine the frequency of weed control often by rule of thumb. This may result in too much weed control or the failure to carry out the weed control during periods in which ecological damage should be kept to a minimum. To be able to formulate rules about weed control the weed growth characteristics and the hydraulic and hydrological aspects involved must be understood.

The discharge capacity depends on the dimensions of the water course. Therefore weed control should already be considered when new water courses are being desiged. At that stage it is easy to modify the design in order to minimize the cost of construction and the recurrent annual maintenance costs. A water course with a greater capacity than required, needs less weed control and the exact timing of weed control is less important. The resulting benefits include the possibility to carry out the maintenance with less expensive equipment, and less man power, or at a time more favourable for nature conservation.

Scheduling weed control using expected discharges and permissible flow rates

I outlined a procedure in which the weed control for a water course can be determined (Fig. 74). A one-dimensional model of the unsaturated zone was used to compute the expected discharges during the year (Fig. 78). The expected discharge is lower in summer and this enables a fair amount of weed obstruction to be tolerated. The permissible flow rate depends on the dimensions of the water course, together with the weed growth characteristics (obstruction) and an acceptable water level in the water course. The frequency of weed control in two water courses in the Poelsbeek area was determined (Tables 30 and 31). Weed control twice a year is feasible for this catchment, using expected discharges in summer to have a recurrence interval of five years.

Approach used to evaluate weed control

The purpose of the evaluation was to identify the options which meet the objectives stipulated by the decision maker. The cost-benefit analysis and the multi-objective decision analysis used in Chapter 8 require various input data, but some of these data are not very precise. The cost-benefit analysis showed that in general the financial losses are much less than the cost of weed control. The cost of weed control is unlikely to have introduced large errors in the analysis. The only uncertainty is the cost assigned to weed control carried out once a year. A single annual clearing would mean that a large amount of weeds would need to be removed in one go, which might require special equipment. If this pushes up the cost too much, the once-yearly weed control option becomes less attractive.

The accuracy of the input data is more questionable for the multi-objective decision analysis than for the cost-benefit analysis. Furthermore, this type of analysis must be regarded as a first step towards an integrated approach for the scheduling of weed control. The data required on the ecological impact were largely subjective, i.e. based on expert judgement. In the field the impact of weed control on terrestrial and aquatic vegetation can differ greatly within a small area. Therefore I used very general criteria in Section 8.4.2 to quantify this impact. It is essential for the ecological impact to be monitored by the water boards over a number of years. That is the only way in which the water boards will obtain local information on the phenomenon. Different weed control strategies could be tried out, based on local experience and the results given in Section 7.3.2 or Chapter 8. My findings show that the danger of obtaining unwanted hydrological conditions, especially too high groundwater levels and high water levels in the water courses, is not as great as one was afraid of (SIEFERS, 1985).

9.2 Recommendations for further research

Various worthwhile avenues for future research were revealed by the research.

Regional hydrological modelling

Hydrological models are used at various levels of complexity for many practical problems related to regional water quantity and quality. Models are used for problems on a small scale to a very large scale covering the whole of the Netherlands. The question if a particular model can be used with the adopted schematization and the processes included in the model, is often not investigated. For instance to use an unsaturated zone model with a time step of 10 days is questionable (QUERNER, 1993). The same applies to the use of steady state groundwater models for situations where the hydrological system never reaches a steady state.

Refining the combined surface and groundwater flow model MOGROW

Preferential flow must be included in the schematization of the unsaturated zone. Figure 49 indicates how this could be done. It is most important to collect more field data so that this concept can be used with confidence in a wide range of situations in the Netherlands.

The schematization of the hydrological system for the model MOGROW requires a fair amount of 'professional judgement'. For instance the spacing of the nodal points (for the saturated groundwater flow) determines the area represented by each node, required for input of data and obtaining results. Subregions are considered for simulating the unsaturated zone, but the required number of subregions is difficult to assess. The research or modelling objective stipulates the required accuracy of the output. Research should be carried out to translate this accuracy of output to the necessary schematization. This scaling problem from reality to model input data is required for all the processes considered in the model MOGROW.

The combined surface and groundwater flow model MOGROW is in general a valuable tool for analysing water management problems in the Netherlands and abroad. The model is suitable for research problems, but is too complex for most hydrologists. The user-friendliness will have to be improved so that water managing authorities will be able to use it. This means incorporating the model within a framework of tools which handle input data, analyse output data and display the important results easily. Such a framework must be flexible and able to handle a wide range of situations and sources of input data.

Weed obstruction

The amount of weed obstruction in a water course plays an important role in the conveyance capacity. The required capacity should be maintained during the growing season, which means doing a number of times weed clearing. The weed obstruction over the growing season is therefore very important and should be monitored. Various hydrological conditions, such as seepage or leakage and water quality, play an important role. Data on weed obstruction should be collected from different regions representing the major growth conditions of weeds in the Netherlands.

Scheduling weed control using expected discharges and permissible flow rates

The procedure developed to estimate the time of weed control is not yet generally applicable in the Netherlands, because not all of the required data are available. The discharge characteristics were determined for sandy soils and only for three water table fluctuation classes (Gt). We still need to find out if the water table fluctuation classes (including sub-classes) is a unique circumstance that produces a certain discharge characteristic, or whether it is possible to have major variations in discharge characteristics within a sub-class. To make the weed control scheduling procedure generally applicable, the discharge characteristics have to be determined for all the major soils present in the Netherlands. Then it will be possible to estimate when weed control is necessary for a range of typical situations, characterized by water courses of different dimensions, maximum hydraulic gradients and minimum freeboard. The programme I developed needs to be made more user friendly before it can be applied by the water boards.

Evaluating weed control

To be able to evaluate weed control by means of a cost-benefit analysis or multi-objective decision analysis, reliable input data from various sources are required. The damage in terms of financial loss caused by the extreme hydrological conditions should be quantified as accurately as possible. The estimation of these losses for agriculture should consider the various farming situations and major soils in the Netherlands. The modelling approach used by PEERBOOM (1990) would be a good way of handling this aspect.

The ecological impact of different weed control strategies should be monitored in the field. Here too, data should be collected in representative catchments for typical hydrological situations and soil types.

Maintenance integrated in the design stage of water courses

An economical evaluation should be carried out, in which weed control is considered within the framework of designing new water courses (or a re-design as part of land reallocation projects). Water courses in the Netherlands are designed to cope with high discharges occurring in winter. In the summer period the discharges are much smaller, but due to weed growth the conveyance capacity will be reduced. A water course with a greater capacity than required in winter needs also less weed control. A larger water course is more expensive to construct; more land is required; the excavation is more and a greater area needs protection (grass cover). A cost-benefit analysis should be carried out taking all these aspects into account. WESSELING (1982) carried out such an analysis. Combining his method with the procedure to determine the time of weed control (Fig. 74) should be carried out.

Samenvatting

Deze studie richtte zich op de verantwoording van het maaionderhoud in waterlopen. Hierbij is gebruik gemaakt van hydrologische modellen. Jaarlijks worden de waterlopen een aantal keren geschoond door de waterschappen in Nederland. Dit 'klein' onderhoud is nodig om de watertransportfunctie van de waterlopen gedurende het groeiseizoen te waarborgen en is daardoor van invloed op de waterstanden in de waterlopen en op de grondwaterstanden. Een hydrologisch model dat dit onderhoud dient te analyseren zal de stroming van water in het oppervlaktewater en in de grond moeten beschrijven. Dit is gerealiseerd door het grondwatermodel SIMGRO te koppelen aan het oppervlaktewatermodel SIMWAT. Het gekoppelde model, MOGROW, is in het Hupselse Beekgebied geverifieerd. De stromingsweerstand in de waterloop en de groei van waterplanten zijn in dit onderzoek betrokken. Hiervoor zijn in een proefgebied in Twente (stroomgebied Poelsbeek en Bolscherbeek van 6400 ha) metingen uitgevoerd. Voor verschillende frequenties van onderhoud, zijn berekeningen gedaan. De resultaten van deze berekeningen zijn geëvalueerd door middel van een kosten-baten analyse en een beslismodel met meervoudige doelstellingen. De belangrijkste onderdelen van deze studie worden hieronder besproken.

Gekoppeld oppervlaktewater- en grondwaterstromingsmodel MOGROW
Het model MOGROW werd ontwikkeld om daarmee de waterbeweging in de verzadigde zone, de onverzadigde zone en het oppervlaktewater te beschrijven. Het model is gebaseerd op fysisch-mathematische grondslagen en is toepasbaar in situaties waarbij de hydrologie verandert. De verzadigde zone bestaat uit een quasi driedimensionaal eindige elementen model. De onverzadigde zone wordt ééndimensionaal beschouwd. Complexe processen zoals oppervlakkige afstroming, schijngrondwaterspiegels, hysteresis en preferente stroming zijn hierin opgenomen. De hoofdwaterlopen in een gebied worden als een netwerk van leidingvakken en knooppunten in het oppervlaktewatermodel geschematiseerd. Kunstwerken zoals stuwen, gemalen, schuiven en duikers zijn speciale leidingvakken of knooppunten. De overige waterlopen worden samengevoegd tot een bak die gekoppeld is aan het netwerk.

Het model MOGROW is geverifieerd in het Hupselse Beekgebied. Hierbij is gebleken dat het niet moeilijk is om met het model de variatie in grondwaterstanden te berekenen, maar de variatie in afvoeren is veel moeilijker om goed te beschrijven.

Stromingsweerstand

De stromingsweerstand uit de formule van Manning is berekend met behulp van metingen in het veld en in een meetgoot. Een goede oplossing werd verkregen als voor het berekenen van de stromingsweerstand alleen het open gedeelte van het dwarsprofiel werd gebruikt. De afvoer door het begroeide deel is zo gering, vergeleken met het open water deel, dat het te verwaarlozen is. De stromingsweerstand k_M^o voor het onbegroeide deel varieerde tussen 30–34 $m^{1/3} \cdot s^{-1}$.

Numeriek model om de stroomsnelheidsverdeling te berekenen

Het model SIMVEL is gebruikt om de stroomsnelheidsverdeling in een dwarsprofiel met daarin begroeiing te berekenen. Hieruit wordt het debiet berekend, waarna met behulp van de formule van Manning de stromingsweerstand wordt afgeleid. Deze berekende stromingsweerstanden waren nagenoeg gelijk aan de direct uit veldmetingen berekende weerstanden. Het model SIMVEL is ook gebruikt om het debiet in een waterloop te berekenen, door gebruik te maken van een beperkt aantal veldmetingen. Het programma berekent dan de stroomsnelheid voor die delen van het dwarsprofiel waar geen metingen beschikbaar zijn. Met deze berekende stroomsnelheidsverdeling over het gehele dwarsprofiel wordt het debiet bepaald. Voor een waterloop met een breedte van ongeveer 6,5 m gaf een berekening met SIMVEL op basis van 18 metingen uit het veld nog een goede schatting van de afvoer (oorspronkelijk 50 metingen).

Obstructie door vegetatie

De obstructie door de vegetatie van het natte profiel en de taluds is gedurende het groeiseizoen gemeten. De obstructie is zowel gemeten in een traject zonder enig onderhoud alsook in een traject met twee keer onderhoud. De resultaten van alle metingen zijn in drie klassen verdeeld op basis van de gemiddelde waterdiepte (fig. 62 en 63). Voor 1990 en 1991 verschilde de gemeten relatieve obstructie maar weinig.

Berekening van tijdstip en frequentie van onderhoud met behulp van verwachte afvoeren en toelaatbare debiet

Het benodigde onderhoud in een waterloop kan worden berekend met behulp van de verwachte afvoer en de groei van de vegetatie. De afvoeren zijn hiervoor berekend met behulp van een ééndimensionaal grondwatermodel van de onverzadigde zone. Het resultaat van deze berekeningen is de variatie in afvoer over het groeiseizoen. Het toelaatbare debiet voor een waterloop wordt berekend voor het onbegroeide deel, gebruikmakend van de stromingsweerstand k_M^o. Als de verwachte afvoer groter is dan het toelaatbare debiet, is onderhoud nodig. Het onderhoud is berekend voor twee waterlopen in het stroomgebied van de Poelsbeek, door gebruik te maken van de berekende afvoeren voor drie grondwatertrappen (Gt III, IV en VII).

De riooloverstorten van het stedelijk gebied zijn ook meegenomen in de berekeningen. Geconcludeerd is dat beide waterlopen één onderhoudsbeurt gedurende het groeiseizoen nodig hebben, maar op een verschillend tijdstip (tabel 31).

Methode om het onderhoud te evalueren

De evaluatie van onderhoudsschema's is getest voor het stroomgebied van de Poelsbeek en Bolscherbeek. Berekeningen zijn uitgevoerd met het model MOGROW voor zeven weerjaren, waarbij verschillende tijdstippen en frequenties van onderhoud in zeven scenario's zijn aangenomen. De resultaten van deze berekeningen (hoge waterstanden, hoge grondwaterstanden en hoge stroomsnelheden) geven een schatting van de schade door wateroverlast. In eerste instantie is een kosten-baten analyse uitgevoerd. Uit deze berekeningen bleek dat voor de meeste weerjaren één keer onderhoud per jaar voldoende is.

Het beslismodel ELECTRE II is toegepast om bovendien ecologische aspecten mee te nemen. Op deze manier is een afweging mogelijk met meerdere doelstellingen, zoals onderhoudsschema's die rekening moeten houden met de belangen van óf de landbouw óf de natuur. Voor landbouw is één onderhoudsbeurt vóór het begin van de winter een acceptabele mogelijkheid. Ook de schema's met één beurt gedurende het groeiseizoen en één voor de winter is een goede mogelijkheid. Voor een minimale ecologische schade is de beste keuze het alternatief: onderhoud van de bodem en één talud gedurende het groeiseizoen. Hiermee wordt voldoende afvoercapaciteit gegarandeerd en een deel van de vegetatie blijft gespaard om als schuilplaats te dienen voor vogels, reptielen en andere fauna.

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List of symbols

Symbol	Definition / Description	units
a,b	constants	–
A	wetted cross-sectional area	m^2
A_c	catchment area	m^2
A_e	area of finite element	m^2
A_i	area of node i	m^2
A_j	area of land use j per subregion	m^2
A_m	minimum required wetted area	m^2
A_o	wetted area of open water part	m^2
A_p	projected area of the i^{th} plant in direction of current	m^2
A_s	wetted area for high flow stage	m^2
c	vertical resistance	d
C	Chézy's coefficient of boundary roughness	$m^{1/2}\cdot s^{-1}$
C_e	concord index for ELECTRE2	–
C_d	drag coefficient for the vegetation	–
D	thickness of aquifer	m
D_e	discord index for ELECTRE2	–
D_i	drag force on a plant	N
d_r	thickness of root zone	m
d_p	thickness of resistant layer	m
E	evapotranspiration	$m\cdot d^{-1}$
E_p	potential evapotranspiration	$m\cdot d^{-1}$
f	Darcy-Weisbach friction factor	–
F_a	factor to account for different cross-section shapes (water depth and base width)	–
F_c	reduction factor for specific discharge to account for large catchment areas	–
F_w	reduction factor for flow rate to account for weed growth	–
F_z	factor to account for the variation in side slope	–
g	acceleration due to gravity	$m\cdot s^{-2}$
h	hydraulic head	m
h_r^*	average depth of the groundwater level below soil surface in a subregion	m
h_i	groundwater or surface water level in node i	m
h_j	water level in adjacent node j	m
h_k	top of resistant layer	m
h_p	pressure head	m
h_r	reference groundwater level	m
h_t	perched water table head	m
h_w^s	water level or invert level of a subsystem of the surface water	m
I	bed slope	–
k	hydraulic conductivity	$m\cdot d^{-1}$

Symbol	Definition / Description	units
k_a	extinction coefficient	–
k_e	boundary roughness height	m
k_s	Nikuradse's sand grain roughness height	m
K	von Kármán's constant	–
k_M	boundary roughness coefficient, being n^{-1}	$m^{1/3} \cdot s^{-1}$
k_M^e	equivalent roughness coefficient	$m^{1/3} \cdot s^{-1}$
k_M^o	roughness coefficient for unobstructed part	$m^{1/3} \cdot s^{-1}$
k_M^r	so-called reference roughness coefficient	$m^{1/3} \cdot s^{-1}$
L	length of channel reach	m
L_{ij}	length of section between nodes i and j	m
L_m	mixing length	m
L_w	distance between water courses	m
n	coefficient of roughness	$s \cdot m^{-1/3}$
N	number of plants (Eq. 14)	–
N	number of days the groundwater level exceeds the reference groundwater level (Eq. 62)	–
p	number of summations of interaction groundwater and surface water, being $\Delta t_g/\Delta t_s$	–
p^*	weight level of strong concordance	–
p^o	weight level of average concordance	–
p^-	weight level of weak concordance	–
P	wetted perimeter of cross-section	m
P_n	net precipitation (precipitation minus interception)	$m \cdot d^{-1}$
P_s	rainfall from sprinkling	$m \cdot d^{-1}$
q_a	horizontal flow above the resistant layer to the surface water system	$m \cdot d^{-1}$
q_c	upward flux from phreatic level to the underside root zone	$m \cdot d^{-1}$
q_e	extractions for public water supply and sprinkling	$m \cdot d^{-1}$
q_i	percolation through resistant layer	$m \cdot d^{-1}$
q_l	leakage	$m \cdot d^{-1}$
q_p	expected specific discharge for recurrence interval	$l \cdot s^{-1} \cdot ha^{-1}$
q_s	drainage to channel system per unit length	$m^2 \cdot d^{-1}$
q_w	drainage to secondary and tertiary water courses	$m \cdot d^{-1}$
q^*	weight level for strong discordance space	–
q^o	weight level for weak discordance space	–
Q	flow rate	$m^3 \cdot s^{-1}$
Q_a	all fluxes from adjacent layers, the unsaturated zone, interactions with the surface water and the extractions	$m^3 \cdot d^{-1}$
Q_c	upward flux	$m^3 \cdot d^{-1}$
Q_d	external inflow for surface water system, such as drainage	$m^3 \cdot s^{-1}$
Q_{ji}^*	flow rate calculated in previous iteration	$m^3 \cdot s^{-1}$
Q_{ji}	flow from node j to node i (surface water)	$m^3 \cdot s^{-1}$
Q_{ji}	flow from node j to node i (groundwater)	$m^3 \cdot d^{-1}$
Q_l	boundary flow dependent on the groundwater level (leakage and drainage to large channels)	$m^3 \cdot d^{-1}$

Symbol	Definition / Description	units
Q_m	maximum permissible flow rate	$m^3 \cdot s^{-1}$
Q_n	flow rate for a section without weed growth	$m^3 \cdot s^{-1}$
Q_o	flow rates from nodes j to node i at time t	$m^3 \cdot s^{-1}$
Q_p	maximum expected discharge	$m^3 \cdot s^{-1}$
Q_t	constant fluxes such as drainage to the secondary and tertiary systems, capillary rise and extractions	$m^3 \cdot d^{-1}$
Q_u	additional discharges, such as effluent from sewage treatment works or sewer emergency overflow	$m^3 \cdot s^{-1}$
R	hydraulic radius	m
R_o	hydraulic radius for unobstructed part	m
R_s	strong outranking relationship	–
R_w	weak outranking relationship	–
s	physical soil unit	
S	hydraulic gradient	–
SOS	sum of daily exceedances of a typical reference groundwater level	cm·d
S_i	storage area for node i of surface water system at time t	m^2
S_k	range of scale for discord	–
S_m	maximum hydraulic gradient	–
t	time	s or d
T	transmissivity	$m^2 \cdot d^{-1}$
T_{ij}	transmissivity matrix for horizontal flow (for a phreatic aquifer dependent on the water-bearing part of the layer)	$m^2 \cdot d^{-1}$
T_w	period of weed growth	d
v	velocity	$m \cdot s^{-1}$
v_a	average velocity for wetted area	$m \cdot s^{-1}$
v_l	laterally averaged velocity	$m \cdot s^{-1}$
v_b	velocity at a distance y_b from the boundary	$m \cdot s^{-1}$
v_i	average approach velocity to the i^{th} plant	$m \cdot s^{-1}$
v_*	local boundary shear velocity	$m \cdot s^{-1}$
V	moisture storage in root zone for each land use	m
V_{eq}	equilibrium moisture storage of the root zone	m
V_s	saturated soil moisture storage in the root zone	m
W	weighting parameter between the time levels t and $t+\Delta t$	–
W_r	relative weed obstruction	–
W^+	sum of weights for criteria where i is preferred to j	–
$W^=$	sum of weights for criteria where i and j are equivalent	–
W^-	sum of weights for criteria where j is preferred to i	–
y	water depth	m
y_b	distance from the boundary	m
y_n	normal depth of flow	m
y_m	maximum acceptable water level	m
z	angle of side slope (Eq. 12)	–
z	vertical coordinate with origin at the soil surface, directed positively upwards (Eq. 36)	m
$\{A\}$	vector of polynomial terms	

Symbol	Definition / Description	units
$[B]$	matrix of polynomial terms as evaluated at the nodes of an element	
$[K_e]$	fluid element 'rigidity'	
$[K_r]$	resistance and storage matrix	
$[K_t]$	assembled fluid element rigidity matrix	
$[L_e]$	interpolation matrix	
$\{F_e\}$	external forces on element	
$\{F_t\}$	assembled vector of all external forces	
$\{h\}$	vector with unknown water levels	m
$\{T\}$	vector of all the known terms	
α	relative evapotranspiration	–
α_i	generalized coordinates	
α_s	geometry factor depending on the shape of the water table	–
α_v	coefficient to express the non-uniform velocity distribution in the cross-section	–
γ	specific weight of liquid	$N{\cdot}m^{-3}$
Δh	change in hydraulic head or water level over time step	m
Δh_t	change in perched water table over time step	m
Δt_g	time step for groundwater system	d
Δt_s	time step for surface water system	s
ΔV	change in moisture storage of root zone over time step	m
ε	eddy viscosity	$N{\cdot}s{\cdot}m^{-2}$
μ	storage coefficient unsaturated zone (below root zone)	–
μ_p	storage coefficient for unsaturated part below resistant layer	–
ρ	fluid density	$N{\cdot}s^2{\cdot}m^{-4}$
τ_a	shear force per unit area on the channel boundary	$N{\cdot}m^{-2}$
τ_n	shear force at boundary node n	$N{\cdot}m^{-2}$
Υ_s	drainage resistance	d
ϕ_m	excess of dissipation	$N{\cdot}s^{-1}$

List of abbreviations

CAPSEV	- Capillary rise in soil profiles (model)
CPU	- Central processing unit (unit for computational time of computers)
CULV	- Computer program to estimate the equivalent roughness coefficient for a water course with a number of culverts (model)
ELECTRE	- Elimination and choice translating algorithm (multi-objective decision method)
FEMSAT	- Finite element method for saturated groundwater flow - unsteady state (model)
FEMSATS	- Finite element method for saturated groundwater flow - steady state (model)
Gt	- Groundwater table fluctuation class
LGN	- Landelijke bodemgebruikskartering Nederland (Land use data bank)
MOGROW	- Modelling groundwater flow and the flow in surface water systems (SIMWAT and SIMGRO models combined)
NAP	- Normaal Amsterdams Peil (reference level in the Netherlands)
SHE	- Système Hydrologique Européen (model)
SIMFLOW	- Simulation of unsaturated soil water flow (model)
SIMGRO	- Simulation of groundwater flow and surface water levels (model)
SIMPRO	- Simulation of flow in surface water networks and the unsaturated zone (SIMWAT and SIMFLOW models combined)
SIMVEL	- Simulation of velocity distribution (model)
SIMWAT	- Simulation of flow in surface water networks (model)