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HYDROLOGICAL MODELLING OF EXTREME FLOODS IN SWEDEN

Joakim Harlin

COVER PHOTOGRAPH

The Stornorrfors hydropower dam in the Umeälven river, discharging 890 (m^3/s) through the spillways during the spring flood in June 1979. The photograph was taken by Bengt Johansson, Vattenfall.

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Abstract

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The design precipitation and snowpack magnitude are the most important prescriptions in the Swedish guidelines for design flood determination. Consequently, the regional design precipitation sequence and all suggested correction factors to the design rainfall sequence (area, altitude and season) are crucial. In application, the most important factors are: choice of hydrological model, model calibration and selected climate period. These factors can lead to an uncertainty in the order of plus/minus 20 % on the design flood peak in catchments with good quality data and good model performance. In catchments with poor data quality, the uncertainty in design flood simulation will be larger. The corresponding uncertainty on reservoir water stage development will depend on local conditions.

Key words
Hydrological modelling, extreme floods, design flood determination, model structure, model calibration, climatic variability, sensitivity analyses, frequency analyses.

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PREFACE

This thesis is based on the following papers, which will be referred to in the text by the Roman numerals I - VII.

I	Bergström, S., Lindström, G. and Harlin, J. (1991) Spillway design floods in Sweden, Part 1: New guidelines. Hydrological Sciences Journal. (Submitted.)
II	Harlin, J. and Lindström, G. (1991) Spillway design floods in Sweden, Part 2: Sensitivity analysis of inflow to single reservoirs. Hydrological Sciences Journal. (Submitted.)
III	Lindström, G. and Harlin, J. (1991) Spillway design floods in Sweden, Part 3: Sensitivity analysis of water stage development in a multi-reservoir system. Hydrological Sciences Journal. (Submitted.)
IV	Harlin, J. (1991) Development of a process oriented calibration scheme for the HBV hydrological model. Nordic Hydrology, Vol. 22, 15 - 36.
V	Harlin, J. and Kung, C.-S. (1992) Parameter uncertainty and simulation of design floods in Sweden. Journal of Hydrology. (In press.)
VI	Harlin, J. (1992) Modelling the hydrological response of extreme floods in Sweden. Nordic Hydrology. (Submitted.)
VII	Harlin, J. (1989) Proposed Swedish spillway design guidelines compared with historical flood marks at Lake Siljan, Nordic Hydrology, Vol. 20, 293 - 304.

Paper I is a summary of the Swedish design flood guidelines, given in full by the Swedish Committee for Design Flood Determination (Flödeskommittén, 1990). In the thesis, paper I serves as an introduction to papers II and III. It was prepared by Sten Bergström in cooperation with Göran Lindström and myself. The modelling system used for design flood simulation was computer-coded mainly by Göran Lindström. Papers II and III are a result of team-work between the authors.

In paper V, I am responsible for the initial ideas, the computer coding and simulations. Chen-Shan Kung assisted during the project and helped with the preparation of the manuscript.

CONTENTS

ABSTRACT

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The Swedish guidelines for design flood determination are examined with emphasis on sensitivity analyses of modelling aspects. The research is mainly based on conceptual hydrological modelling with the HBV model. An automatic calibration methodology is presented and discussed. The influence of model calibration, model structure and climatic variability on the design flood simulation as well as the preset meteorological and hydrological conditions suggested in the guidelines are studied. The thesis also includes a discussion of the annual exceedance probability of the Swedish design floods by comparisons with observed extreme floods and frequency analyses.

The design precipitation and snowpack magnitude are the most important prescriptions in the Swedish guidelines for design flood determination. Consequently, the regional design precipitation sequence and all suggested correction factors to the design rainfall sequence (area, altitude and season) are crucial. In application, the most important factors are: choice of hydrological model, model calibration and selected climate period. These factors can lead to an uncertainty in the order of plus/minus 20 % on the design flood peak in catchments with good quality data and good model performance. In catchments with poor data quality, the uncertainty in design flood simulation will be larger. The corresponding uncertainty on reservoir water stage development will depend on local conditions.

The HBV model was found appropriate for extreme flood modelling. However, using the model for design flood calculation requires special attention to calibration of the snow and recession parameters.

Design flood modelling in a hydropower-developed river system shows that it is difficult to assess the integrated effects of extreme precipitation, snowmelt, soil moisture status and reservoir operation in advance. For some reservoirs the spring flood is most critical, while the autumn flood is more severe for others. The design flood generated by simulating the total upstream area to a dam is more severe than that from simulating the local catchment only. The results indicate that the critical flood and water stage development increases, the further down a hydropower system one gets.

In practical applications of the Swedish guidelines for design flood determination, uncertainty in several of the computational steps influences the final results. But, these factors could often be varied considerably without changing the conclusion on whether a particular dam could meet the requirements of the new guidelines or not.

Key words: Hydrological modelling, extreme floods, design flood determination, model structure, model calibration, climatic variability, sensitivity analyses, frequency analyses.

1. INTRODUCTION

1.1 Background

1.1.1 Dams and public safety

Dams are built to serve a number of purposes, for example: irrigation, water supply, hydropower and flood mitigation. Dams dampen and store floods, but they are also potentially dangerous for the downstream area. A sequence of events that can initiate a dam failure can be identified (Figure 1). These events can be external such as floods, earthquakes and upstream dam failure, or internal such as changes in soil or concrete properties or construction defects. They would normally not be critical for the dam. At extreme situations, however, for example at high inflow rates, a rapid rise in pool level could lead to overtopping and dam failure. The rise in pool level during a flood will depend on the magnitude of the flood, storage capacity, initial pool level, reservoir operation and spillway capacity of the dam. It is also affected by the availability of the turbines. The consequences will depend on various exposure factors. These include season, flood warning lead time and time of the day. Figure 1 is an illustration of the interactions between these factors for an embankment dam. Embankment dams are particularly sensitive to overtopping.

The total number of high hazard dams in the world, in the event of failure, is more than 150 000. Many of these structures have not functioned as planned. As a rough estimate, 1 % of all dams have failed and more than 8 000 people have died in these disasters (Jansen, 1983). The single most common cause has been overtopping due to insufficient spillway capacity (35 % of the incidents). The corresponding figure for USA is 40 % (Jansen, 1983). Reiter (1988) compiled the causes of dam failures as:

Foundation failure, seepage and settlement	43 - 50 %
Insufficient spillways and incorrect dam operation	27 - 37 %
Poor construction material	7 - 14 %
War	1 - 3 %
Unknown	2 - 18 %

As a result of dam failures and incidents, many countries have enacted new or revised laws and guidelines for design, supervision and operation of dams and reservoirs. An International Commission of Large Dams (ICOLD) was founded in 1929. This organization is instrumental in collecting and sharing knowledge gained by professionals on dam design and construction from all countries of the world. Design flood determination and operational flood control was given special attention at ICOLDs sixteenth international congress on large dams during June 1988 in San Francisco, USA (ICOLD, 1988).

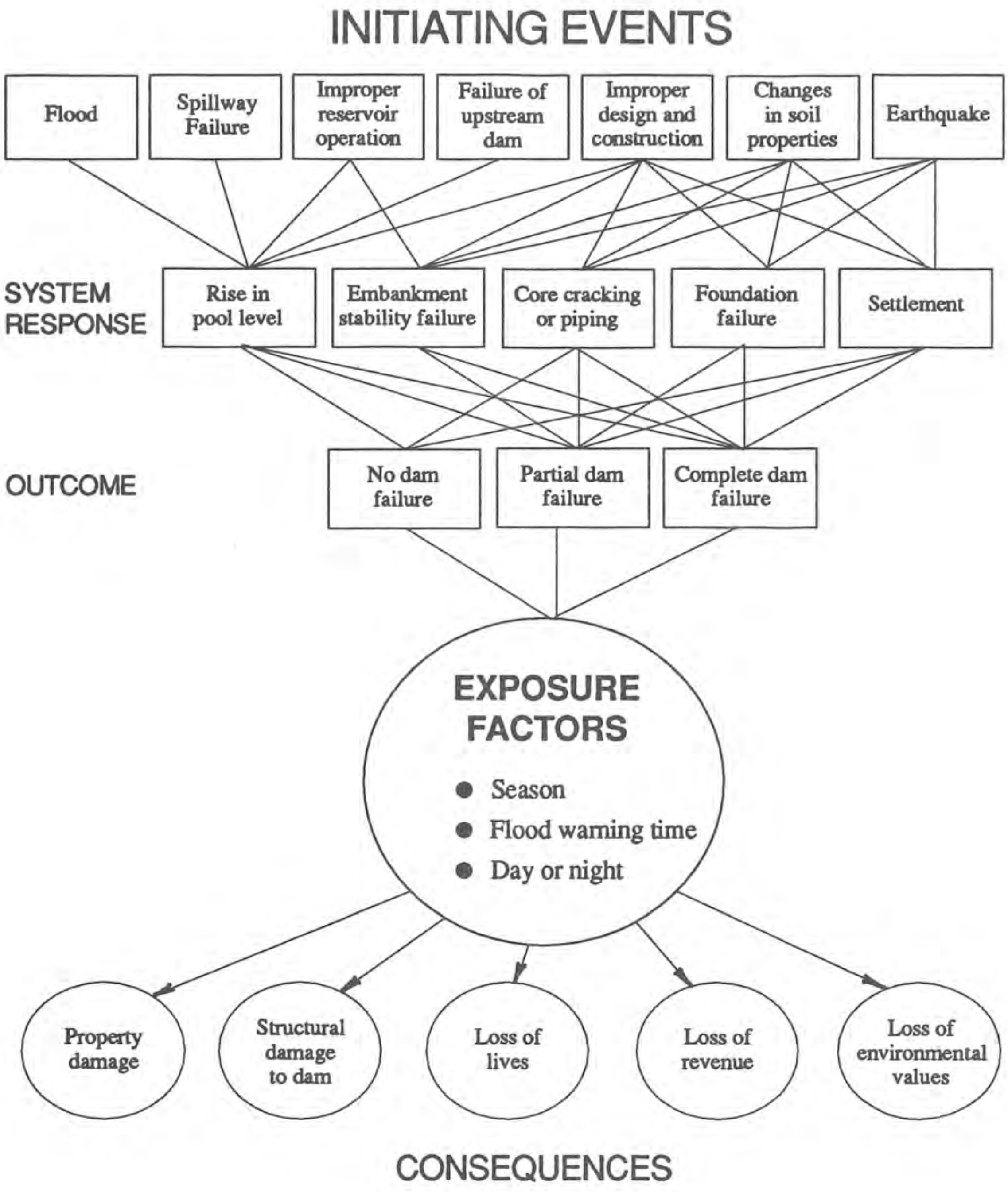

Figure 1. *Event sequence diagram, leading to downstream damage, for an embankment dam (modified after Bowles, 1987).*

1.1.2 Hydropower in Sweden

Sweden has a total area of 449 964 km^2 and thus is one the largest countries in Europe. However, its population density is low. The total population of Sweden is about 8.6 million, which gives only 19 people per square kilometer on an average. About 65 % of the area are forest land, 7 % are alpine (above the forest line), 11 % are farming or grazing land, 8 % are marsh and peat land and 1 % is urban areas. The remaining 8 % of Sweden are covered by more than 100 000 lakes (National Atlas of Sweden, 1990).

As a whole, Sweden is a country of lakes and rivers with vast forests between.

Sweden has a humid climate with distinct seasons. In general, the highest precipitation occurs in the western mountain range, where some places experience as much as 1500 mm/year. Precipitation is also high in the south-western coastal region; about 1000 mm/year. The interior and lowland parts of the country are fairly dry and have less than 600 mm/year. The largest evaporation is in Southern Sweden, roughly 400 mm/year, and decreases northward (SMHI, 1979; Alexandersson et al., 1991). Most of the yearly evaporation takes place during the summer months June, July and August. In winter, the predominant precipitation in Central and Northern Sweden, falls as snow.

The total electricity consumption in Sweden is of the order of 130 TWh per year, which is high in an international perspective; only Norway and Canada have a larger consumption of electricity per capita. Hydroelectric plants cover about half of the production, nuclear plants about 45 % and thermal power produces the remaining 5 % of the electricity (The Swedish Power Association, 1990). These figures vary from year to year depending on demand, electricity exports or imports and the hydrological situation in the country.

Most of the main rivers flow from NW to SE and all but four are developed for hydropower and are highly regulated; for example, the regulation in the Luleälven river in Northern Sweden corresponds to an active storage capacity of 72 % of the total annual runoff. In principal, the regulation strategy is to transfer the runoff in spring, summer and autumn to the winter, when the electricity demand is highest. More than one third of the hydropower produced comes from water stored for shorter or longer periods in the reservoirs. The production in mid winter is up to ten times that corresponding to the natural runoff of the rivers. There are about 1000 hydropower stations in the Swedish rivers and about 130 dams with a height larger than 15 meters, of which 80 % are embankment dams (The Swedish Power Association and the State Power Board, 1987).

The hydropower developed rivers in Sweden are extremely complex systems of reservoirs and power stations, often with different owners (Figure 2). For the majority of them there is a river regulation enterprise. These enterprises are responsible for the building, administration, operation and regulation of the hydropower reservoirs. They are owned by the hydropower producing companies, whose power stations benefit from the river regulation (Vattenregleringsföretagen, 1991).

No major dam accidents have to date occurred in Sweden. However, some smaller dam failures have been experienced; for example the dams at Sysslebäck in July 1973 and Noppikoski in September 1985. In both these cases the cause was extreme floods and insufficient spillways.

In early 1985, a committee with the task of developing new guidelines for design flood determination, was established. The members of the committee represented both the hydroelectric power industry and governmental agencies (the Swedish Meteorological and Hydrological Institute, SMHI). The final guidelines were presented in the summer of 1990 (Flödeskommittén, 1990). They are described and discussed in Section 1.3.

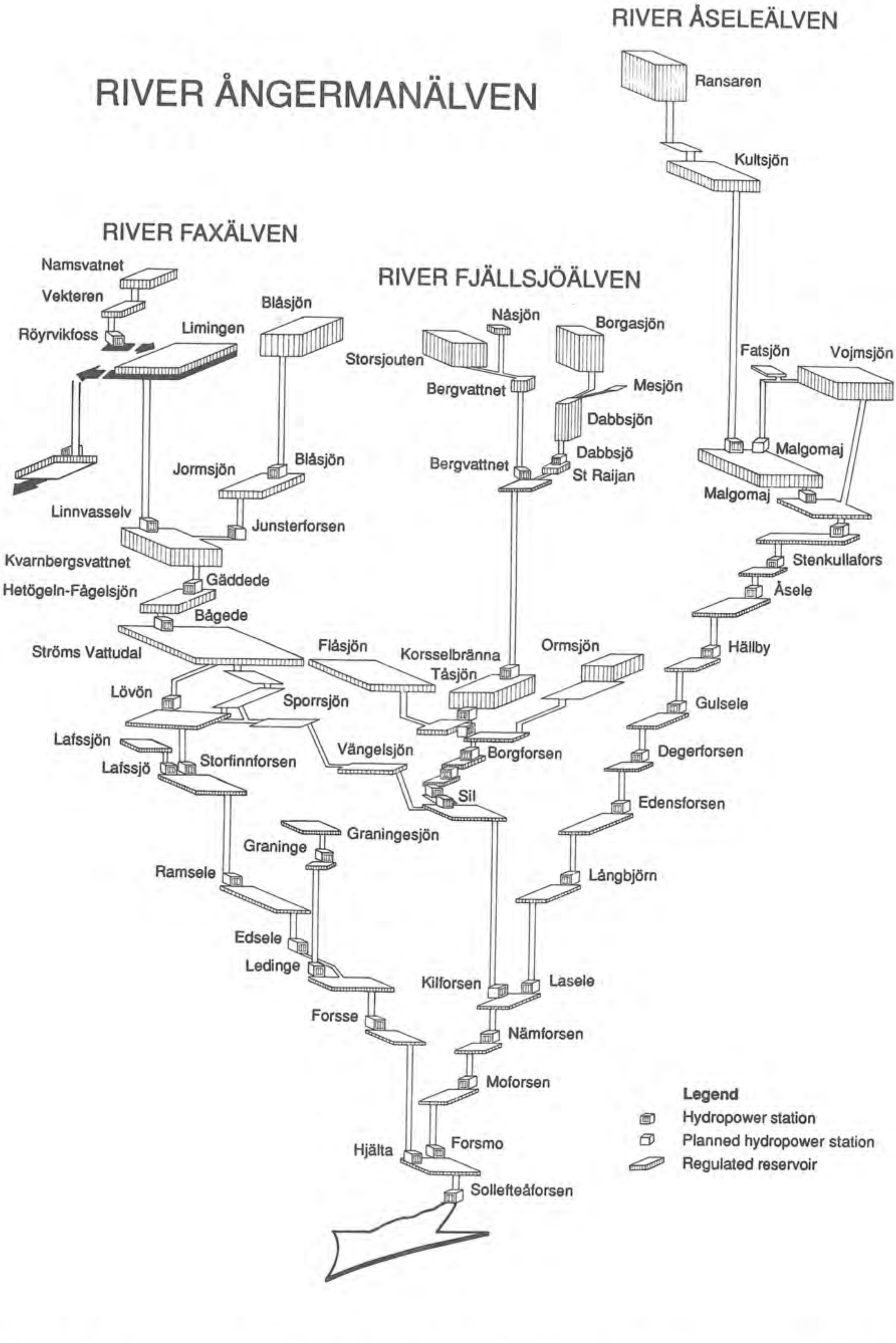

Figure 2. *Schematic of the hydropower development in the Ångermanälven river in Northern Sweden (Source: The Ångermanälven River Regulation Enterprise).*

1.2 Design flood determination

1.2.1 General principles

The selection of criteria for designing a dam will determine the final construction. From an engineering point of view, several criteria have to be decided, for example loadings and environmental influences to which the dam and its foundation will be subjected, physical characteristics of the construction and foundation materials, economical constraints and levels of risk. Several of these aspects are well defined and uncertainty factors can be used to describe the extreme conditions forming the design criteria. This is common practice for analyses of loads from dead weight of material, silt loads, pore pressure or stresses in construction materials. However, one of the most important loadings and influences on a dam, the floods, is more difficult to assess.

Extreme floods occur seldom per definition, and sometimes no large floods at all have been recorded at a potential dam site. Today, it is obvious that substantially larger floods than those recorded are possible and will probably occur sometime during the lifetime of a dam. The difficulty is to estimate the magnitude of these rare floods. This question has always bothered engineers and hydrologists when designing spillway capacity, reservoir storage and regulation rules. In English the critical flood for a dam is called the "design flood", in German the "Bemessungshochwasser", in French the "(débit de) la crue de project" and in Swedish the notion "dimensionerande flöde" is used.

In the early hydropower development in Sweden, the most common way to determine the design flood was to use the highest observed flood or water level recording at the site. This was sometimes supplemented with observations at other stretches of the river or from neighbouring basins. As the observation records grew longer, knowledge of the behavior of extreme floods increased. Larger floods than those recorded were considered possible and a safety factor of 10 to 20 percent was added to the highest observed flood, thus forming a design flood (Melin, 1963).

Extreme floods in Sweden are in general caused by critical combinations of flood generating factors such as heavy rainfall, saturated soils and intense snowmelt in the catchment rather than by extreme precipitation alone (Lindström, 1990). To describe the probability of each factor or the joint probability of the factors causing an extreme flood is difficult. Two main philosophies are today common for calculating design floods. The first approach is based on statistical analysis of the streamflow. This is usually done by frequency analysis of annual flood maxima. In doing so, the integrated effect of the different flood generating factors are studied directly and the annual exceedance probability of the flood is obtained.[1] The other method is based on a deterministic approach. The flood generating factors are maximized and converted to a flood by use of a unit hydrograph or by more sophisticated hydrological models. International practices for

[1] The annual exceedance probability of a flood is often expressed in the inverse form as a return period. For example, a flood with an annual exceedance probability of 0.001 or 1/1000 is referred to as a 1000 year flood.

design flood determination, based on these two philosophies, are discussed in Sections 1.2.2 and 1.2.3 respectively.

A third philosophy based on risk analysis and calculation of the consequences of a hypothetical dam failure, is sometimes practiced. With this method, estimates of the costs and revenues of the dam and the economical development in downstream areas have to be made for the entire lifetime of the dam. The design flood is not calculated directly, it is selected through an iterative computation process, in which costs and risks are evaluated, at the lowest risk-cost alternative (NRC, 1985). Critique against this approach is that factors that cannot be measured in economic terms are involved in the risk-cost analysis; such as loss of human life, social losses and environmental impacts. Furthermore, it relies on flood frequency analysis and predictions of the cost-benefit development during the lifetime of the dam and how values in the society will change during this period. The risk-cost analysis approach is still being developed throughout the world and there is an ongoing debate on whether a value for loss of human lives should be included (Cantwell and Murley, 1988).

To determine a design flood calculation philosophy means that the acceptable degree of safety for reservoirs or multi-reservoir systems has to be established. In practice, the potential risk for human life, psychological, ethical and political considerations predominate the judgement. Therefore, the approach to the problem differs substantially from one country to another and sometimes even within one country.

1.2.2 Statistical methods

Statistical methods focus on the random variation of a set of flood observations, and only indirectly on the physical processes that produce them. One statistical method is based on envelope curves from plotting of maximum flood peaks observed in a hydrologically homogeneous region against catchment area. This curve is then considered as the upper limit of expected flood peaks for the considered region. By adopting a frequency distribution to the data, multiplication factors for calculating design floods of various recurrence intervals are obtained. This methodology is commonly practiced in Southern Africa, for example in South Africa, Lesotho, Swaziland, Namibia, Botswana, Mozambique and Zimbabwe (ICOLD, 1992; Kabell, T. C. 1986 and 1988).

The most commonly used statistical method for extreme flood determination is however, flood frequency analysis (Figure 3). Annual flood maxima (Q_{max}) are assumed to be a sample of independent data of a total population belonging to a frequency distribution. The observations are used to select a frequency distribution and fit the parameters. The frequency distribution is then used to extrapolate floods to any low probability. Regional and historical information can be incorporated in the analysis to produce more accurate estimates of extreme floods. Many different frequency distributions are used for this purpose, the most common ones in application are the Gumbel, log Pearson III and log-normal distributions (Cunnane, 1989).

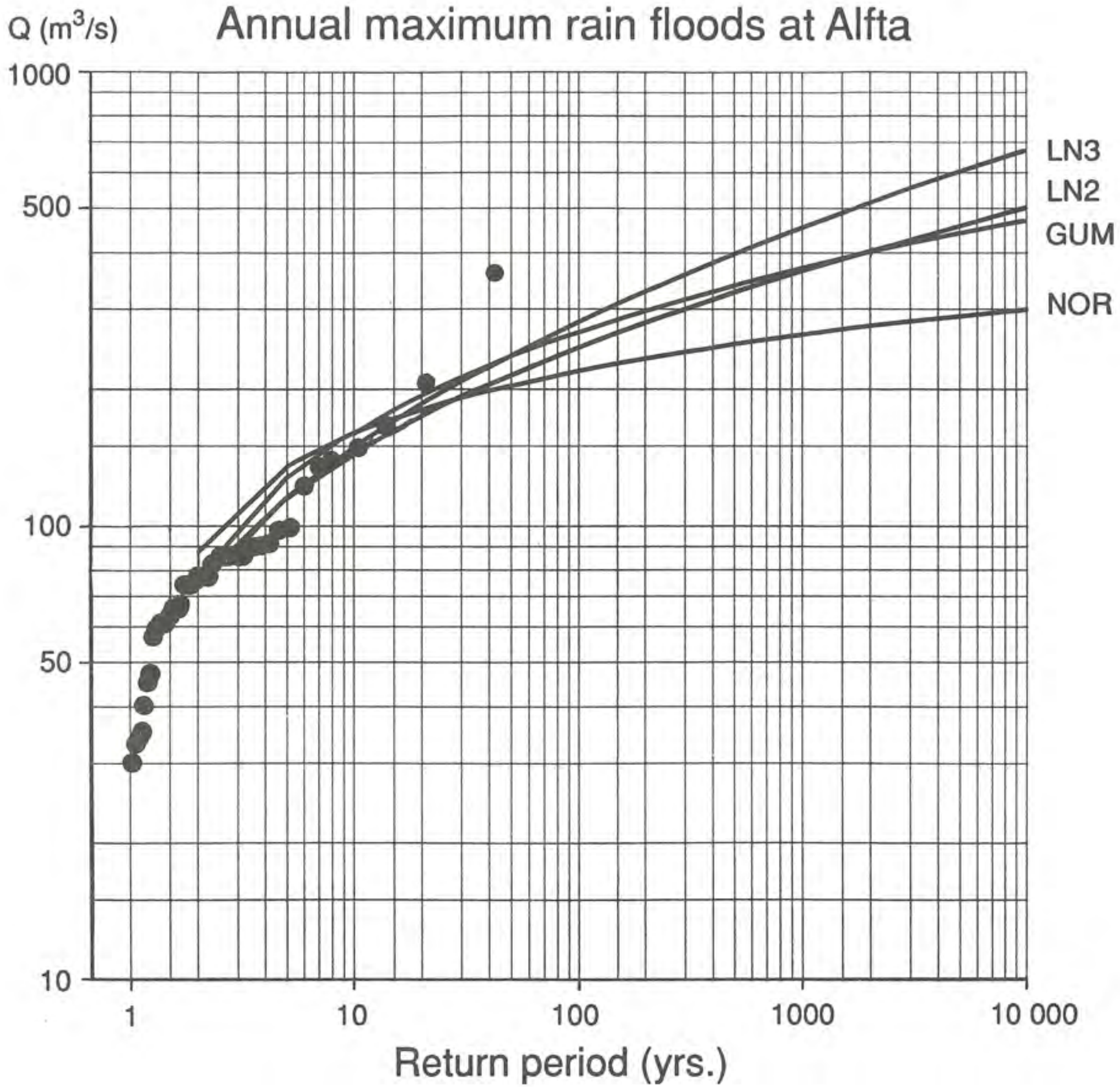

Figure 3. *Example of a frequency diagram, showing annual maximum rain floods at Alfta (1951 - 1991) in the Voxnan river of Central Sweden. The four frequency distributions; three and two parameter lognormal, Gumbel and normal distribution are depicted. The observations are plotted with the Weibull plotting position formula.*

Frequency analysis has the advantage of being simple to apply and of specifying the return period (T), for any given flood magnitude. However, the method suffers from several limitations. The sources of error can be summarized as follows (see, for example, Cunnane, 1989; Kite, 1988; Yevjevich, 1972; NRC, 1988; and Haan, 1977):

- inability of the frequency distribution to reproduce the population Q_{max}-T relation,
- incorrect choice of frequency distribution,
- bias in parameter estimation procedure,
- sampling error due to the fact that parameters are estimated from a finite sample, and
- the available record may not be a truly random sample from the required population.

Furthermore, when estimating design floods by flood frequency analysis, an extrapolation to floods having a recurrence interval much greater than the period of record is made. In most countries, available runoff records are at the longest about 100 years long, which means that an extrapolation in the order of 100 times has to be made in order to obtain the design flood. Some examples of the uncertainties related to design flood determination through frequency analysis are given by Wang (1988).

In Finland, frequency analysis based on the Gumbel frequency distribution is used for extreme flood determination. For high hazard dams a flood return period of 5 000 - 10 000 years is recommended as design flood. For less potentially dangerous dams return periods in the range 100 - 1500 are used (Vattenstyrelsen, 1985). Examples of other countries which have used this method for design flood calculation are Austria (Widdman, 1988), Switzerland (Biedermann et al., 1988), former U.S.S.R. (Kritsky and Menkel, 1967), Czechoslovakia (Broza, 1988), Germany (DVWK, 1989), Spain (Garcia 1988) and China (Shi-Qian 1987; Pan and Teng, 1988).

In some countries frequency analysis is used for determining design floods to low hazard dams, or used in conjunction with deterministic methods, for example in Sweden (Flödeskommittén, 1990), Norway (NVE, 1986), Great Britain (NERC, 1975), Australia (ANCOLD, 1986) and USA (NRC, 1985; USWRC, 1986).

In France, the Gradient of Extreme Values (GRADEX) method was developed to avoid the uncertainties due to extrapolation of runoff records (Duband et al., 1988). Frequency analysis is applied to precipitation data instead of runoff data. Floods are generated by converting precipitation amounts to runoff volume after reduction for soil moisture deficits. The soil moisture deficit is assumed to decrease with increasing precipitation. The Gumbel frequency distribution is used to describe both the precipitation and the flood frequencies. The method has for the past twenty years been used by the large governmental owned power company Electricité de France, but it is not part of the legislation.

1.2.3 Deterministic methods

The most commonly practiced deterministic method is to calculate the probable maximum flood (PMF) by a combination of the probable maximum precipitation (PMP) with critical hydrological runoff conditions. The PMP and PMF concepts are defined by NRC (1985) as:

> *"Probable Maximum Precipitation (PMP): Theoretically, the greatest depth of precipitation for a given duration that is* ***physically possible*** *over a given size storm area at a particular geographical location at a certain time of the year.*
>
> ...
>
> *Probable Maximum Flood (PMF): The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions*

*that are **reasonably possible** in the region."* [2]

In the Australian guidelines on design floods for dams the PMF is defined slightly differently. Their definition reads:

> *"Probable Maximum Flood, (PMF): The flood hydrograph resulting from PMP and, where applicable, snowmelt, coupled with the worst flood-producing catchment conditions that can be **realistically expected** in the prevailing meteorological conditions." (ANCOLD, 1986.)* [2]

The definition of PMP is more precise than that of PMF. This is due to the fact that PMP is estimated with the physical laws of the processes involved in extreme precipitation, where all factors are maximized. PMF on the other hand has to be generated by combining extreme rainfall with hydrological factors such as soil moisture deficit, ground water storage, snowmelt etc. Furthermore, the computation of the PMF involves the use of a runoff model, which in turn adds to the uncertainty in the concept. Figure 4 illustrates the principle of the deterministic method of computing the design flood hydrograph.

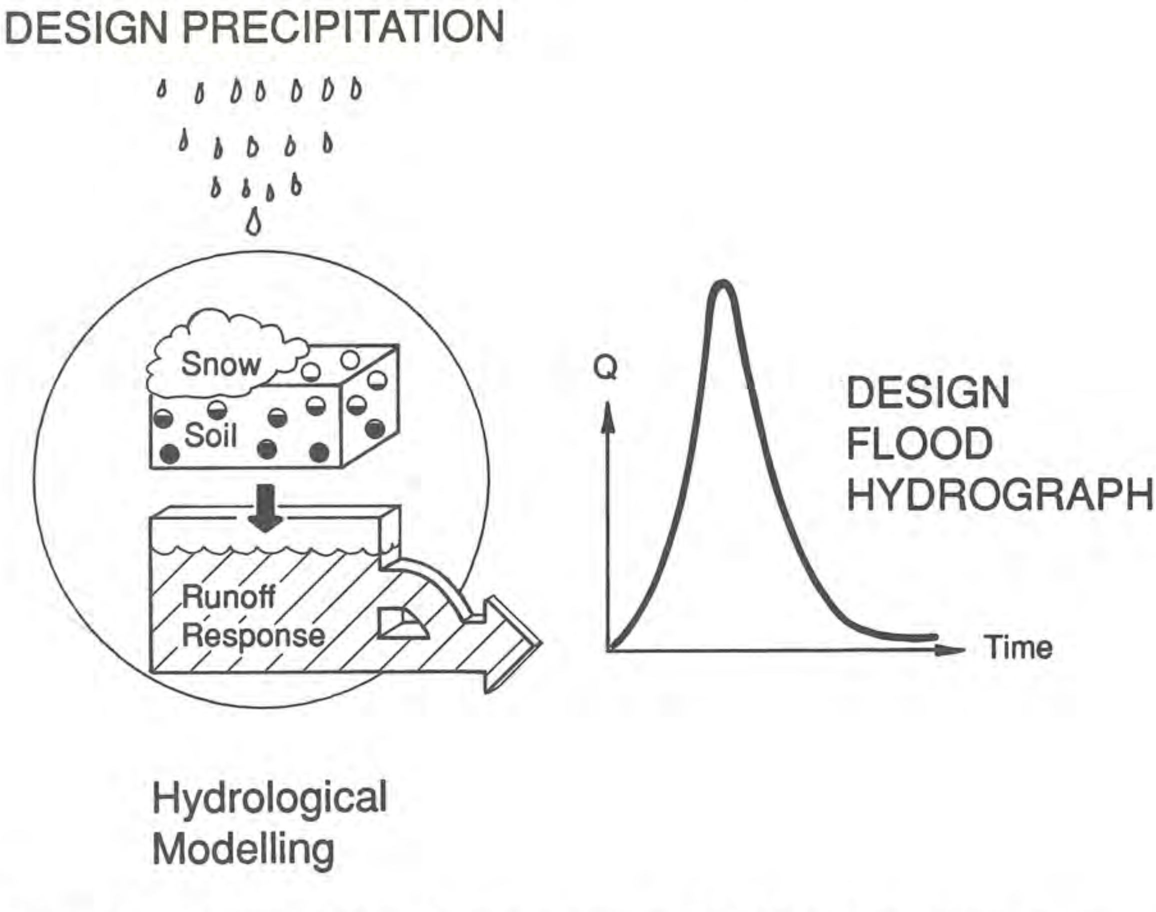

Figure 4. Illustration of the deterministic method of design flood determination.

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[2] My bold face marking.

There is no universal method for computing the PMF, each country has its own approach. The computational steps for deriving and routing the PMF hydrograph through a reservoir, for Federal use in the USA are given by NRC (1985) as follows:

"1. *dividing drainage area into subareas, if necessary;*
2. *deriving runoff model;*
3. *determining PMP using criteria contained in NOAA Hydrometeorological Report series;*
4. *arranging PMP increments into logical storm rainfall pattern;*
5. *estimating for each time interval the losses from rainfall due to such actions as surface detention and infiltration within the watershed;*
6. *deducting losses from rainfall to estimate rainfall excess values for each time interval;*
7. *applying rainfall excess values to a runoff model of each subarea of the basin;*
8. *adding to storm runoff hydrograph allowances for base flow of stream, runoff from prior storms, etc., to obtain the synthesized flood hydrograph for each subarea;*
9. *routing of flood for each subarea to point of interest;*
10. *routing the inflow through the reservoir storage, outlets, and spillways to obtain estimates of storage elevations, discharges at the dam, tailwater elevations, etc., that describe the passage of the flood through the reservoir."*

Several of the computational steps for deriving the PMF from the PMP are influenced by the judgement of the hydrologist conducting the calculation. Due to the element of subjectivity in this process the PMF is not as strictly defined as the PMP.

An advantage with the deterministic method of design flood calculation is that information from one basin can be transferred to another and that the whole hydrograph is obtained, not only the flood peak. This enables the engineer to study the water stage development in single as well as in multi-reservoir systems. Hydrological modelling also provides the possibility of studying different combinations of flood generating factors and by sensitivity analysis quantifying the relative importance of each one.

However, the PMF is a theoretically generated and hypothetical flood which can lead to a false feeling of security, that this flood defines an upper boundary that never can be exceeded, which is not true. A flood with a peak flow representing 81 % of the PMF occurred in 1950 in the Little Nemaha river, Nebraska (Bullard, 1986) and in Eastern United States estimated PMFs have been reevaluated, providing new larger PMFs, at many sites during the last three decades (Lave et al., 1990). Another disadvantage with the concept is that there is no corresponding probability of exceedance that can be related to the calculated PMF.

In many countries the PMF is compared with estimates of the 10 000 year flood. But since the PMF normally is so much larger than the highest observed flood, it can be questioned whether a frequency analysis extrapolated to such extremes is possible at all. This issue was examined by the Hydrological Subcommittee of the Interagency Advisory

Committee on Water Data in the USA (U.S. Department of Commerce, 1986). They concluded that *"no procedure proposed to date is capable of assigning an exceedance probability to the PMF or to near-PMF floods in a reliable, consistent, and credible manner."*

The deterministic approach (PMF or similar) is used all over the world for designing high hazard dams, for example in Canada (Kartha, 1988), USA (NRC, 1985), Venezuela (Castillejo and Semeler, 1988), Brazil (Carvalho, et al. 1988), Paraguay (Schulman, et al. 1988), Norway (NVE, 1986), Sweden (Flödeskommittén, 1990), Great Britain (NERC, 1975), India (Mohile and Kathuria, 1988), Thailand (Champa et al., 1988) and Australia (ANCOLD, 1986).

1.3 The Swedish guidelines for design flood determination

1.3.1 Methodology

The Swedish design flood guidelines suggest a classification of dams into two categories, high-hazard and low-hazard, depending on the consequences of a dam failure. High-hazard dams are dams that in the event of failure would cause large risk for: human life, extensive damage on infrastructure or nature and considerable economical damage. Low-hazard dams are classified as potentially dangerous for property and natural values only. The following presentation is limited to a short summary of paper I, and deals only with design flood determination for high-hazard dams in Sweden. A complete description of the Swedish guidelines for design flood determination is found in Flödeskommittén (1990).

The Swedish guidelines propose a deterministic approach, similar to the PMF procedure, with emphasis on critical timing of flood generating factors. They are however, not based on PMP estimates but on observed maximum areal rainfall in combination with extreme snowmelt simulations in a trial and error procedure until the worst flood for a specific system is found. Thus a very unlikely extreme flood, but one within the realm of possibility, is generated.

In the iteration process, a regional 14-day design areal rainfall sequence, adjusted for season, catchment area and altitude, is successively entered into a hydrological model at alternative dates over a ten-year period. During the simulations the recorded rainfall over the catchment for the ten years is used, except for the 14-day period where the rainfall sequence is located. Before spring each year the soil is assumed to be brought to field capacity and the snowpack is replaced by an estimated 30-year snow value. All possible timings of the design precipitation over the ten-year period are tried until the most critical floods are found. This means that some thousands of simulations have to be carried out and that design snowmelt floods in spring and rain floods at other times of the year, or combinations of the two, are computed by one procedure. The extreme flood hydrographs are then routed through the reservoir under study, employing a regulation strategy developed for each dam, to arrive at the design water stage hydrograph (Figure 5).

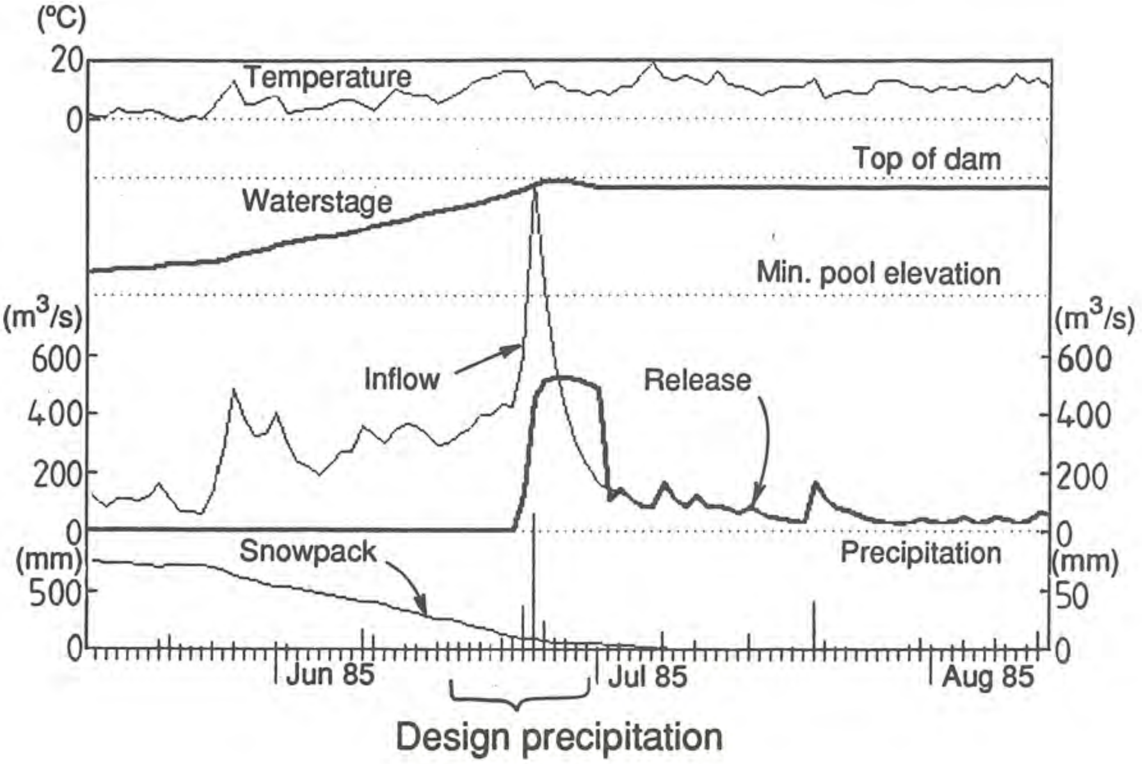

Figure 5. *Example of the most critical inflow simulation and reservoir response in the single reservoir system of Torrön, in the upper part of the Indalsälven river in Northern Sweden (from I).*

For a system of dams, the design procedure becomes more complicated. Releases from upstream reservoirs have to be simulated by a regulation strategy and added to the local inflow to the reservoir being studied (Figure 6). The dam safety criterion is that all dams of the system must be able to withstand the most severe of the simulated inflows.

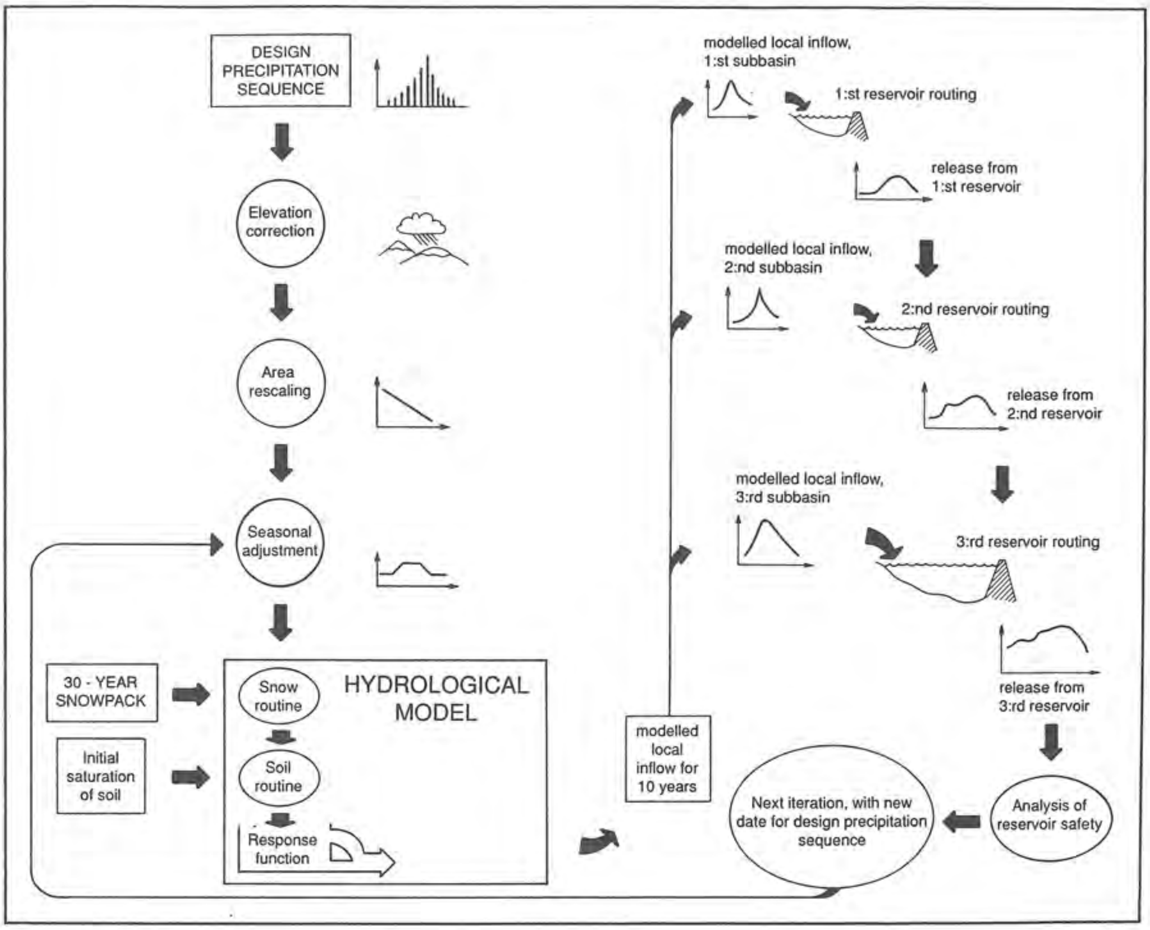

Figure 6. Schematic presentation of flood simulation and routing through a system of reservoirs according to the Swedish guidelines for design flood determination (from I).

1.3.2 Advantages and disadvantages

The Swedish method of design flood determination has similar advantages and disadvantages as the PMF concept in general. The main difference is that observed maximum areal precipitation is used instead of an estimated PMP. Advantages with the approach suggested by Flödeskommittén (1990) are that:

- it prescribes all flood generating meteorological and hydrological conditions in detail for the whole country in one document, which leaves little room for subjectivity in application.

- it can be applied to a complete system of reservoirs and can be used for calculation of snowmelt and rain floods or combinations of the two.

- it is based on critical timing of observed flood generating factors. None of the flood generating factors are extrapolated beyond observations.

- it utilizes all essential and available hydrometeorological data for a region.

Disadvantages with the approach suggested by Flödeskommittén (1990) are that:

- it does not specify the exceedance probability of the generated design floods.
- it involves extrapolation with a hydrological model to floods larger than the observed ones. Consequently, the design floods are sensitive to i) model calibration and ii) choice of model.
- it is sensitive to the characteristics of the climate period used in the design flood modelling.
- it leads to extensive and complex computations when applied to multi-reservoir systems.

During the process of development, the guidelines have been presented and discussed at several national and international symposia and conferences, Table 1. They are presently being applied in reevaluation of existing dams and in the design of new dams in Sweden.

Table 1. Symposia and conferences at which the Swedish guidelines for design flood determination have been presented and discussed.

Symposia/Conferences	Publication
Nordic Hydrological Conference, Reykjavik, 1986	Bergström et al., 1986
Seminar on Dams, The Royal Institute of Technology, 1986	Ohlsson, 1986
Nordic Hydrological Conference, Rovaniemi, 1988	Bergström, 1988a
XVI International Congress on Large Dams, San Francisco, 1988	Bergström, 1988b Bergström and Ohlsson, 1988
Seminar on Dams and Concrete The Royal Institute of Technology, 1988	Harlin, 1988
The Seventh Northern Research Basins Symposium/ Workshop, Illulissat, 1988	Bergström et al., 1988
The Swedish Committee for Design Flood Determination, Review Seminar, Vattenfall, 1988	Flödeskommittén, 1986, 1989
ICWRS Workshop on Risk and Uncertainty in Hydrologic Design, Oslo, 1989	Bergström et al., 1989 Harlin, paper VII
Nordic Hydrological Conference, Kalmar, 1990	Bergström, 1990a
VAST/Vattenfall, Seminar on Dam Safety, 1990	Bergström, 1990b
Dam Safety Symposium, The Royal Institute of Technology, 1990	Sprinchorn, 1990 Ohlsson, 1990
XX General Assembly IUGG: IAHS Symposium, Vienna 1991	Lindström et al., 1991

2. OBJECTIVES

The overall objective of the thesis is to analyse the Swedish guidelines for design flood determination, with emphasis on sensitivity analyses of modelling aspects. The following tasks can be identified:

- to identify the most important factors when applying the Swedish design flood guidelines to single reservoirs and multi-reservoir systems (I, II, III).
- to develop an automatic calibration scheme and study the influence of calibration on the modelling of design floods (II, III, IV, V).
- to describe the influence of the model structure on the modelling of design floods (VI).
- to assess the uncertainty in design flood estimates (V, VI).
- to study the annual exceedance probability of the design floods by frequency analysis incorporating historical flood marks at the Siljan lake (VII).
- to study the influence of climatic variability on the modelling of design floods (II, IV, V).

The automatic calibration scheme was developed with design flood simulations and sensitivity analyses as one application. It is also used for a large number of other model applications.

3. STUDY BASINS

Thirteen basins representing different hydrological regimes in Sweden have been studied (Figure 7). A hydropower reservoir is located in the downstream part of the catchment for a majority of these basins. The northern basins are mountainous, located partly above the forest line. Their unregulated flow has a clear seasonal pattern, with large snowmelt floods in spring (April, May and June), occasional rain floods in summer and autumn (July to November), and base flow during winter (December to March). In the Torrön catchment though, floods are also frequent in winter. The inland basins are to the largest part covered by forest, and their unregulated flow follows a clear seasonal pattern similar to the northern basins.

The Simlången basin on the western coast belongs to a totally different regime with a humid maritime climate producing mainly rain floods throughout the year. The milder climate in Southern Sweden results in shorter winters with little snow and less regular seasonal runoff pattern. The southern basins are partly cultivated and their terrain is in general flat to rolling.

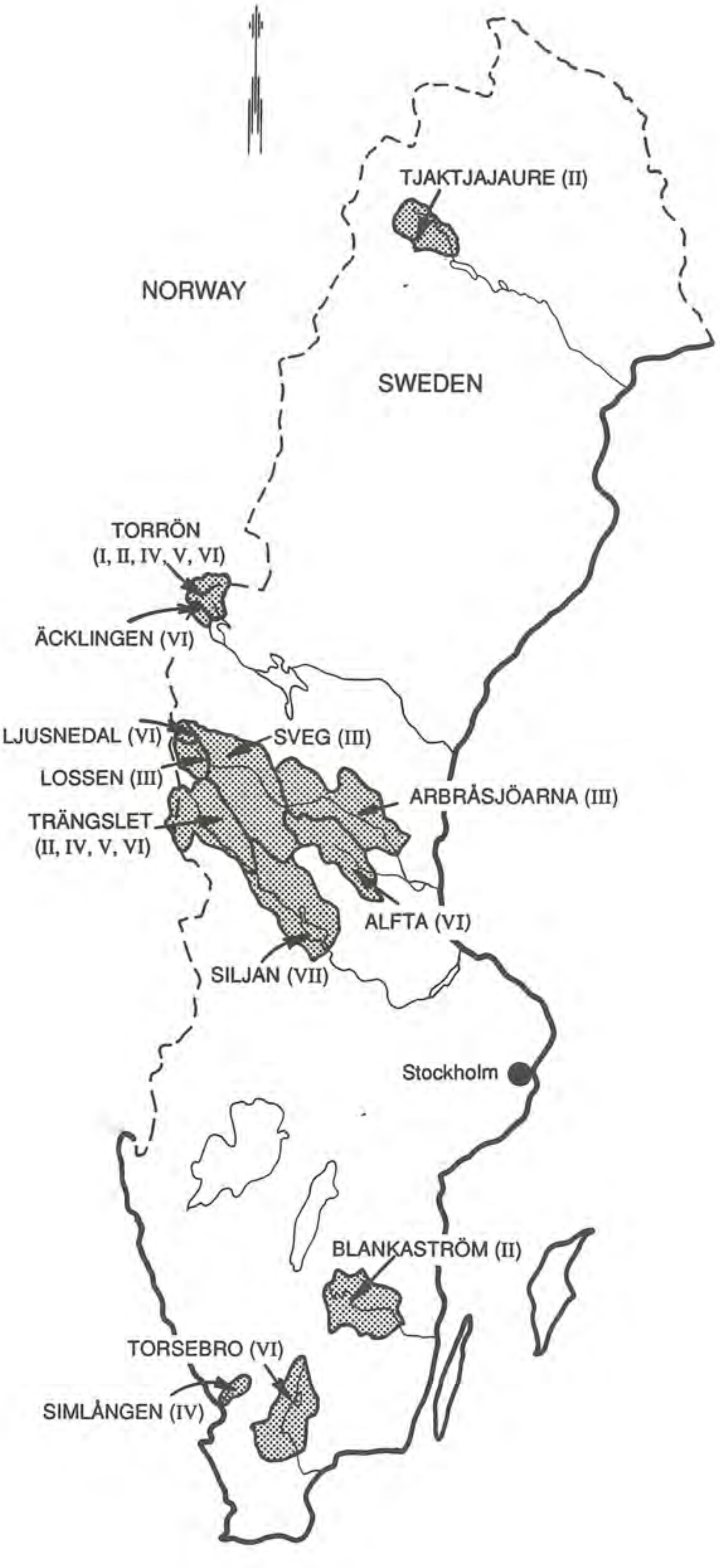

Figure 7. *Locations of all basins studied in the thesis. The Roman numerals refer to the papers where they are discussed.*

4. METHODS

The research reported in this thesis is mainly based on conceptual hydrological modelling. Model development and sensitivity analyses have been carried out with the HBV model (Bergström and Forsman 1973; Bergström, 1976). A process-oriented automatic calibration methodology has been developed. Sensitivity analyses have been performed by repeated simulations after changing one factor at a time and by Monte Carlo experiments. The probability of the design floods have been studied by comparison with frequency analysis, incorporating historical information, of annual maximum water stages at the lake of Siljan.

4.1 Hydrological modelling

4.1.1 The HBV hydrological model

The HBV hydrological model is normally run on daily values of rainfall and air temperature and monthly estimates of potential evapotranspiration. The model contains routines for snow accumulation and melt, soil moisture accounting, runoff generation and a simple routing procedure, Figure 8. It can be used in a distributed mode by dividing the catchment into subbasins. Each subbasin is then divided into zones according to altitude, lake area and vegetation. Bergström (1976) gives a detailed description of the model, only a short description is given in this thesis.

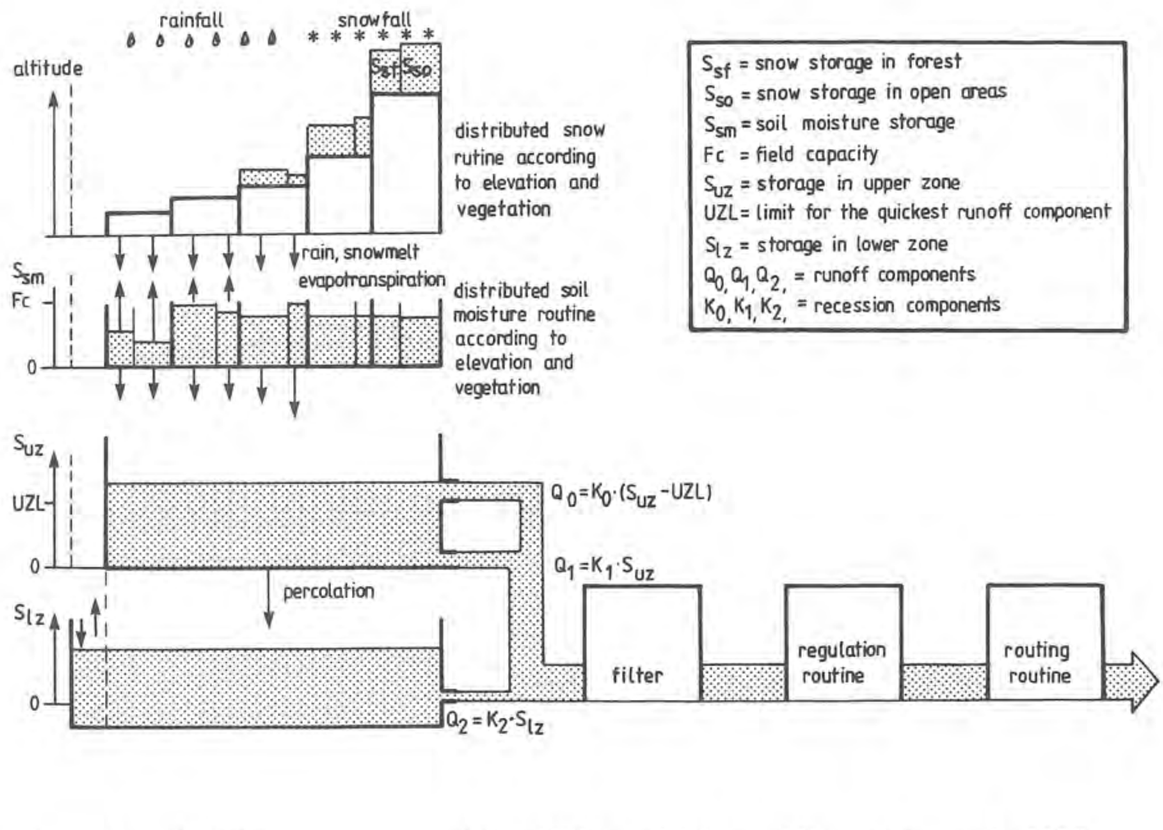

Figure 8. The basic structure of the HBV model (from Häggström et al., 1990).

Snowmelt is calculated separately for each elevation and vegetation zone according to the so called degree-day equation:

$$Q_m(t) = CFMAX \cdot (T(t) - TT) \tag{1}$$

where: Q_m = snowmelt,
CFMAX = degree-day factor,
T = mean daily air temperature, and
TT = threshold temperature for snowmelt.
t = time

Some rain and melt water can be retained in the snow. In the model, a retention capacity of 10 % of the snowpack water equivalent is assumed. The snow routine also contains a general snowfall correction factor (SFCF) which adjusts for systematic errors in observed snowfall and winter evaporation.

Soil moisture dynamics are calculated separately for each elevation and vegetation zone. The percolation of excess water from the soil moisture storage is related to the precipitation and to the computed soil moisture storage as given in Equation 2. Rain or snowmelt generate small contributions of excess water from the soil when the soil is dry and large contributions when conditions are wet.

$$Q_s(t) = \left(\frac{S_{sm}(t)}{FC}\right)^{\beta} \cdot P(t) \tag{2}$$

where: Q_s = excess water from soil,
S_{sm} = soil moisture storage,
FC = soil saturation threshold,
P = precipitation, and
β = model parameter.

Evapotranspiration is computed as a function of the potential evapotranspiration and the available soil moisture, as:

$$E_a(t) \begin{cases} = \dfrac{E_p \cdot S_{sm}(t)}{LP} & if\ S_{sm} \leq LP \\ = E_p & if\ S_{sm} > LP \end{cases} \tag{3}$$

where: E_a = actual evapotranspiration,
E_p = potential evapotranspiration, and
LP = S_{sm} threshold for E_p.

Excess water from the soil and direct precipitation over open water bodies in the catchment area generate runoff through a lumped runoff-response function described by two linked tanks (Figure 8). The lower tank is a linear reservoir, representing base flow. It is filled by percolated water (PERC) from the upper tank plus precipitation over open water bodies, and responds with discharge and evaporation from lakes (Equation 4).

If the excess water from the soil exceeds the percolation capacity (PERC), the upper tank starts to fill. This tank simulates the catchment response to flood events and has two recession coefficients separated with a threshold in storage (Equation 5).

$$Q_l(t) = K_2 \cdot S_{lz}(t) \tag{4}$$

$$Q_u(t) \begin{cases} = (K_0 + K_1) \cdot S_{uz}(t) - K_0 \cdot UZL & \text{if } S_{uz} > UZL \\ = K_1 \cdot S_{uz}(t) & \text{if } S_{uz} \leq UZL \end{cases} \tag{5}$$

where: Q_u = runoff generation from the upper response tank,
K_0, K_1, K_2 = recession coefficients,
UZL = storage threshold between K_0 and K_1,
S_{uz} = storage in the upper response tank,
Q_l = runoff generation from the lower response tank, and
S_{lz} = storage in the lower response tank.

In order to account for the damping of the generated flood pulse in the river, a simple transformation is made. This is a filter with a triangular distribution of weights with the base length MAXBAS. If a lake or a reservoir is located at the catchment outlet it can be modelled separately. There is also an option of using the Muskingum routing routine for modelling the damping out of the generated flood pulse.

The HBV model was originally developed for inflow forecasting to hydropower reservoirs in Scandinavian catchments, but has now been applied in more than 27 countries all over the world (Figure 9). Despite its relatively simple structure, it performs equally well as the best known models in the world (see WMO, 1986).

Some examples of model applications are: hydrological forecasting and computation of design floods in totally about 170 basins in Scandinavia (see, for example, Häggström, 1989; Aam et al., 1977; and Vehviläinen, 1986), modelling the effects of clearcutting in Sweden[3] (Brandt et al., 1988), snowmelt flood simulation in Alpine regions (Capovilla, 1990; Renner and Braun, 1990), hydrological modelling in Arctic permafrost environment (Hinzman and Kane, 1991) and flood forecasting in Central America (Häggström et al., 1990).

[3] This project was carried out with the PULSE model. PULSE is a further development of the HBV model for environmental studies (see Bergström et al., 1985).

Figure 9. *Countries in which the HBV model has been applied.*

4.2 Model calibration

4.2.1 <u>The process-oriented automatic calibration scheme</u>

The philosophy of the developed process-oriented calibration scheme (POC) was to utilize the physical representation of the model components and the experience from manual calibrations (IV). This was done by splitting the calibration period into subperiods, within which one specific process dominates the runoff.

The different subperiods were compiled by combining the observed runoff and temperature data. From the duration curve of observed runoff, characteristic highflow (Q_h) and baseflow (Q_b) discharge limits were estimated. A discharge larger than Q_h was regarded as a flood and a discharge lower than Q_h formed baseflow.

Subperiods dominated by snowmelt floods were found by checking the runoff after cold spells and using Q_h and Q_b to follow the floods and define the start and end of them. Warm periods during which runoff was larger than Q_h formed rainflood subperiods. Base flow periods were compiled by checking when the observed runoff was below Q_b and so on. Figure 10 shows an example of how the different subperiods were discriminated.

The parameters were only evaluated over the subperiods where they were active. In doing so, optimization of only one statistical criteria of total model performance was avoided, and the degrees of freedom for the parameters were reduced providing a more clear picture and of the error caused by each one of the parameters.

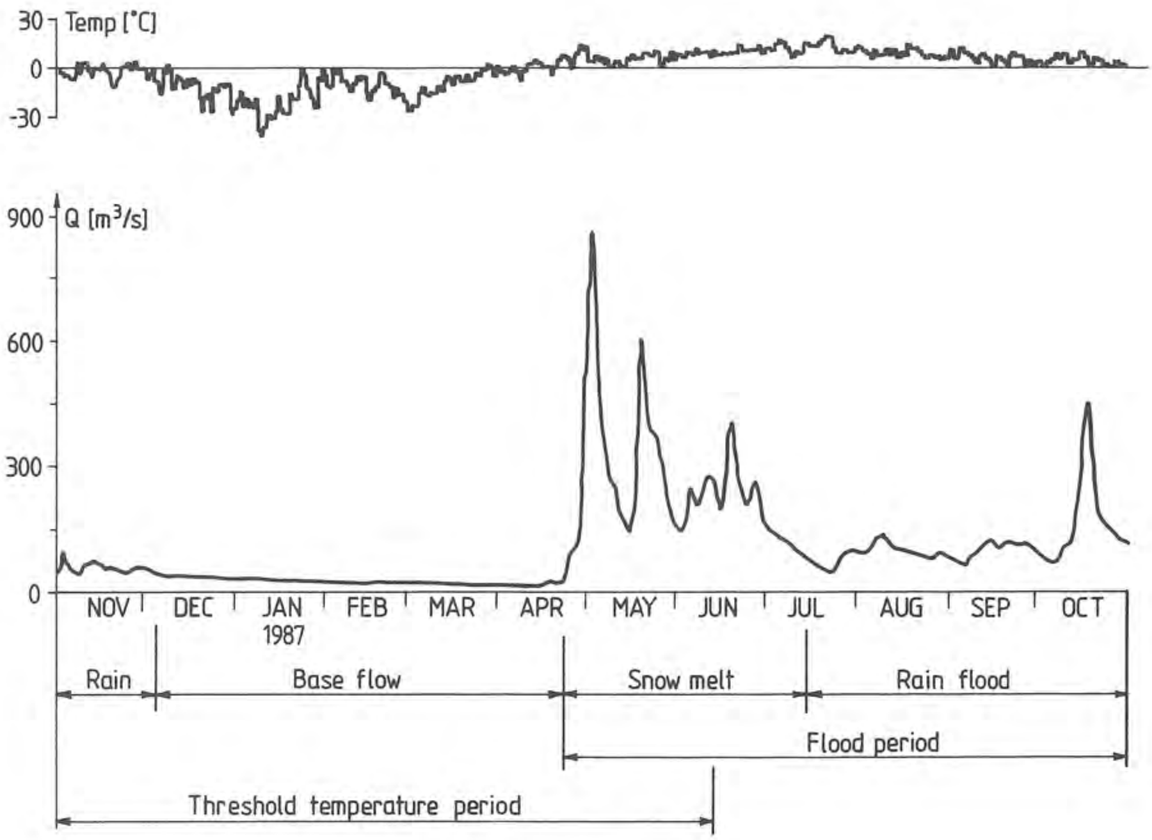

Figure 10. Splitting of the calibration period into subperiods dominated by one process (from IV).

The calibration started with estimates of the parameters K_0, K_1, K_2, UZL and PERC from recession analysis of the observed runoff. MAXBAS was initially set to one day, and the remaining six parameters were initially set to the middle to their respective ranges found by experience from the large number of manual calibrations of Swedish basins. Then an iteration loop was carried out over the whole model, where the parameters were calibrated one at a time in a set order starting with the snow routine, over the soil routine and finally the runoff-response function (Table 2).

For each subperiod an objective function was evaluated. These were the absolute mean accumulated volume deficiency, defined as

$$MAD = \left| \frac{1}{n} \cdot \sum_{t=1}^{n} \left(Q_m(t) - Q_o(t)\right) \right| \qquad (6)$$

where: n = number of time steps in each subperiod,
Q_m = modelled discharge and
Q_o = observed discharge.

This function was used to minimize the volume error of the snowmelt floods. To adjust the phase error of the starting time of snowmelt floods, the mean accumulated absolute error calculated as

$$MABSD = \frac{1}{n} \cdot \sum_{t=1}^{n} |Q_m(t) - Q_o(t)| \tag{7}$$

was found more appropriate. Furthermore, the mean square error MSE defined as

$$MSE = \frac{1}{n} \cdot \sum_{t=1}^{n} (Q_m(t) - Q_o(t))^2 \tag{8}$$

and the efficiency criterion R^2 proposed by Nash and Sutcliffe (1970), expressed as

$$R^2 = 1 - \left(\frac{\sum_{t=1}^{n} (Q_m(t) - Q_o(t))^2}{\sum_{t=1}^{n} (Q_o(t) - Q_{om})^2} \right) \tag{9}$$

where $Q_{om} = \frac{1}{n} \sum_{t=1}^{n} Q_o(t)$ were used.

For a perfect model and perfect data, the simulation will reach an R^2 value of 1.0. A perfect hydrological model provided with perfect data, however, is a hypothetical situation that per definition gives a true simulation of a real hydrological process. If the model application is as poor as for the case of only using the mean discharge, the corresponding R^2 value will be zero.

MSE was used to calibrate parameters active over several subperiods and R^2 was mainly used to evaluate the resulting total model performance. The objective functions were optimized separately for each parameter with Brents parabolic interpolation method (Brent, 1973). The iteration loop over all model parameters continued until the parameters stabilized, i.e. when the R^2 criterion for the whole calibration period stopped alternating. Calibration order, objective function and subperiod for each parameter are given in Table 2.

Table 2. Calibration order, objective functions and subperiods used in the calibration loop (from IV).

Parameter	Objective function	Subperiods
<u>Snow routine</u>:		
SFCF	MAD	Snowmelt floods
TT	MABSD	Below +2 °C *⁾
CFMAX	MSE	Snowmelt floods
<u>Transformation function</u>:		
MAXBAS	R^2	Whole period
<u>Soil routine</u>:		
FC	MSE	Rain floods
LP	MSE	Rain floods
β	MSE	Rain floods
<u>Upper response tank</u>:		
K_0	MSE	All flood periods
K_1	MSE	All flood periods
UZL	MSE	All flood periods
<u>Lower response tank</u>:		
PERC	MSE	Base flow periods
K_2	MSE	Base flow periods

*⁾ The subperiods for TT were all periods where a 14 day moving average of air temperature was below +2 °C.

4.2.2 Automatic calibration performance

The POC scheme yielded as good model performance as the manually calibrated models used for operational flood forecasting at SMHI. In general three - four iteration loops (over the model) were sufficient to find the optimal parameter set. In each loop the objective functions were evaluated about 120 times. Automatic calibration opposed to manual calibration is computer-intensive instead of labour-intensive. Calibration over a ten-year period would typically take between 15 and 20 hours on a 386 processor PC. Model performance in terms of R^2 values, accumulated relative volume errors and volume errors for snowmelt floods only, after POC and manual calibration are given in Table 3. Figure 11 shows an example of resulting hydrographs after manual and POC calibration. Two hydrological years are depicted.

Table 3. Model performance after POC and manually calibrated parameters (from-IV).

Basin	Calibration period (POC)	Verification period (POC)	Total period POC	Man.
1. Torrön				
R^2	85.5 (10 years)	84.1 (10 years)	84.8	79.3
VE	0.9	4.2	1.7	3.4
VS	8.0	11.8	10.0	11.8
2. Trängslet				
R^2	94.7 (8 years)	90.7 (10 years)	92.9	92.0
VE	3.5	9.8	4.1	0.8
VS	5.0	10.3	7.8	7.5
3. Simlången				
R^2	89.2 (10 years)	84.3 (10 years)	86.8	83.8
VE	1.9	7.2	4.8	6.6
VS	5.7	11.7	8.7	7.4

R^2 = model efficiency (%),
VE = volume error (%), accumulated for all time steps, and
VS = volume error (%), accumulated over the snowmelt floods.

The most straightforward way to evaluate calibration results is to compare the quality of the simulations visually. Since the inflow hydrograph contains a large amount of data of a different types, e.g. rain floods, snowmelt floods, baseflow periods etc., it is difficult to find one particular criterion that will objectively show the model performance. The R^2 criterion was chosen, since it shows how well the model describes the observed runoff dynamics. It is also one of the most commonly used criteria for model performance evaluation in hydrology (see WMO 1986 and 1987). The accumulated relative volume error is interesting, because it shows the error in water balance over the studied period. The volume error over the snowmelt floods is important when regulating hydropower reservoirs. In many Swedish rivers these floods constitute the majority of the yearly runoff.

The fact that POC gave slightly better model performance than manual calibration in terms of the criterions should not be overemphasized. As is illustrated in Figure 11, it is difficult to see the difference in quality between simulations giving different values.

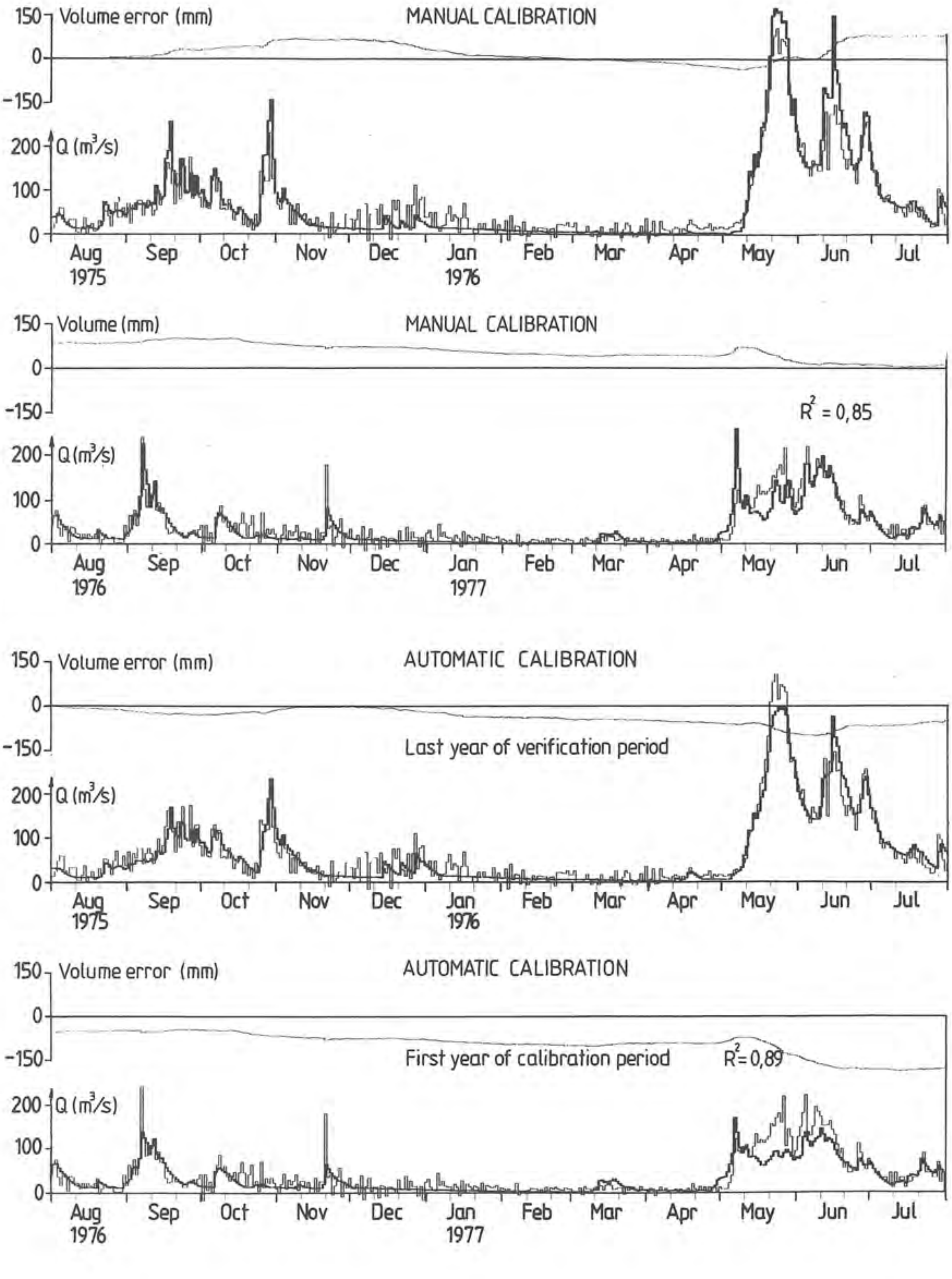

Figure 11. Example of model performance after manual and automatic calibration (POC), for inflow to the Torrön reservoir. Thick and thin curves show computed and recorded inflow, respectively. The automatic calibration period was 10 years. R^2 values refer to plotted periods only (from IV).

To estimate the quality of different calibrations it is useful to split the data into two parts and only calibrate over one data set. The remaining independent period is used to verify the model performance. In this way an overfit of the parameters can be revealed. Furthermore, the calibration and verification periods should be based on data of homogeneous quality and be sufficiently long to cover the hydrological dynamics of the catchment.

Separate calibration and verification periods were used in the POC scheme development. Interpretation of the results should focus the verification period performance. Ten-year verification periods (Table 3) are more than sufficient to cover the runoff patterns of these catchments. The ability of the POC scheme to calibrate the HBV model for alternative lengths of the calibration period is shown in Figure 28, Section 5.5.1. It was found that for certain one- and two-year calibration periods, comparable results to those from a ten-year calibration period were achieved. This indicates that even very short records of streamflow can be useful in water resource planning, if there are longer records of climate data.

However, in general the results of this research indicate that a calibration period of at least four years should be used for good model performance.

4.3 Sensitivity and uncertainty analyses

4.3.1 Classic sensitivity analysis

Classic sensitivity analysis, during which one variable is changed at a time, was carried out. This methodology is simple to apply and understand and is useful for identifying the importance of a single variable on the process studied. It was used to determine the most important prescriptions and modelling steps of the Swedish guidelines for design flood determination.

4.3.2 Monte Carlo experiments

Sensitivity and uncertainty analyses using the Monte Carlo approach were carried out with a similar methodology as that described by Hornberger et al. (1986). Repeated simulations were made using randomly generated parameter combinations. The parameter sets were generated in the following way:

- Step 1. Automatic and manual model calibration over different data periods,
- Step 2. Compilation of the parameter range for each parameter from the results in Step 1,
- Step 3. Random generation of parameter values from a uniform distribution within each individual range defined in Step 2,
- Step 4. Random combination of the parameter values, generated in Step 3, to form parameter sets.

A great number of Monte Carlo simulations were carried out. Each simulation was then classified as producing either acceptable or unacceptable results by checking the corresponding R^2 and volume error criteria of model performance.

The key idea was to compare the frequency distribution of individual parameter values between the acceptable and unacceptable results. If the shapes of the two distributions were clearly different, the output was sensitive to the parameter under study. The shapes of the frequency distributions for acceptable and unacceptable results also indicate if the chance of acceptable or unacceptable results is larger at any interval of the parameter range.

4.3.3 Multiple linear regression

Stepwise forward selection multiple linear regression was used to examine the relationship between the dependent variables (flood peak, Q_{max}, and maximum water stage, Wst_{max}) and the independent variables (the HBV model parameters). This procedure consists of building the regression equation one variable at a time by adding at each step the variable that explains the largest amount of the remaining unexplained variation. Thus the first variable added is the one with the highest simple correlation with the dependent variable. The second variable added is the one explaining the largest variation in the dependent variable that remains unexplained by the first variable added, and so on.

The coefficient of determination r^2 is a measure of how much of the variability in the dependent variable that is explained by the regression. The range of r^2 is from 0.0 to 1.0; it is given by the formula:

$$r^2 = \frac{\textit{Sum of squares due to regression}}{\textit{Sum of squares about the mean}} = \frac{\sum_{i=1}^{n} \left(\hat{Y}_i - \bar{Y}\right)^2}{\sum_{i=1}^{n} \left(Y_i - \bar{Y}\right)^2} \tag{10}$$

where: n = number of observations,
$\hat{Y}_i$ = predicted value of the dependent variable by the regression,
$\bar{Y}$ = mean of the dependent variable, and
Y_i = value of the dependent variable.

4.4 Frequency analyses

Frequency analysis of annual maximum spring and autumn water levels of the Siljan lake were carried out. The five frequency distributions: normal, two and three parameter lognormal, Weibull and the Gumbel distribution were fitted with the method of moments parameter estimation procedure. Chi-square goodness of fit tests and frequency plotting of the observations were carried out to select the best fitting distribution.

For annual maximum spring water levels, three historical flood marks were incorporated in the analysis. The equations for adjusting statistics for historical data defined by USWRC (1982) were used to estimate the frequency distribution parameters. The underlying assumption is that the data from the systematic record is representative of the intervening period between the systematic and historical record lengths. The equations were as follows:

$$W = \frac{H - Z}{N} \tag{11}$$

$$\tilde{M} = \frac{W \cdot N \cdot M + \sum_{i=1}^{z} X_i}{H} \tag{12}$$

$$\tilde{S}^{\,2} = \frac{W \cdot N \cdot \left(M - \tilde{M}\right)^2 + \sum_{i=1}^{z} \left(X_i - \tilde{M}\right)^2 + W \cdot (N - 1) \cdot S^2}{H - 1} \tag{13}$$

where: W = systematic record weight,
H = historical time period,
Z = number of historical floods,
N = number of years in the systematic record,
X_i = magnitude of the i:th historical flood,
M = mean of the systematically recorded floods,
$\tilde{M}$ = mean adjusted for the historical floods,
S = standard deviation of the systematically recorded floods,
$\tilde{S}$ = standard deviation adjusted for the historical floods.

As plotting position for observed water stage data, in the frequency analysis not including historical flood marks, the Weibull plotting position formula was chosen.

$$pp = \frac{N + 1}{m} \tag{14}$$

where: pp = plotting position,
N = number of years,
m = ranking number in order of magnitude.

For frequency analysis incorporating historical flood marks the Bayesian plotting position formula derived by Hirsch and Stedinger (1987) was used.

$$pp = \frac{H + 1}{\tilde{m}} \tag{15}$$

where: $\tilde{m}$ = m for the historical floods,
$\tilde{m}$ = W · m - (W - 1) · (Z + 0.5) for the systematically recorded floods.

5. RESULTS

5.1 Sensitivity of the evaluation procedure

In this section the sensitivity of the preset meteorological and hydrological conditions and the suggested modelling procedure are given. The results illustrate the relative importance of each of the flood generating factors and modelling steps that the Swedish guidelines for design flood deterination suggest. In application, these conditions have to be strictly followed.

5.1.1 Meteorological prescriptions

In general, the largest floods in the design calculation occur when the precipitation sequence is combined with intense snowmelt in the spring. These spring floods are however, not always the most critical for reservoir storage as they sometimes arrive before the filling of the reservoir (II, III). The selected areal rainfall as input to the simulation is the most important component of the guidelines. Changes in the design 14-day areal precipitation sequence had a great impact mainly on flood peaks but also on water stage development (Figures 12 and 13).

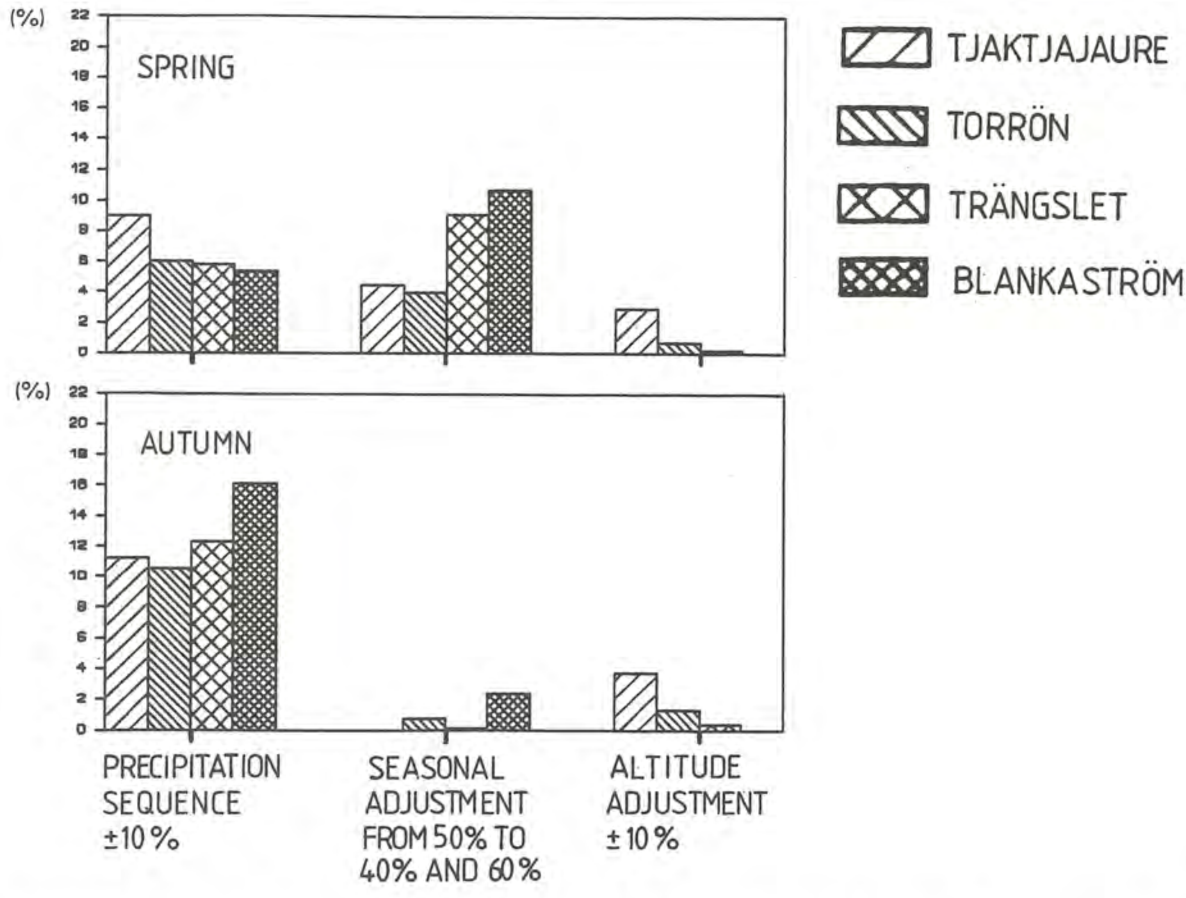

Figure 12. The largest absolute sensitivity of highest flood peaks due to changes in the design precipitation (from II).

Seasonal adjustment of the rainfall was found to be important in spring, but less important in autumn (after August 1st). The explanation is that the largest spring floods in the south occur earlier in the year, when the seasonal reduction is large. In autumn the design floods occurred before the seasonal reduction of the precipitation had any large effect, hence the lower sensitivity. Altitude correction was only crucial for catchments above or partly above the reference altitude. Flödeskommittén (1990) found this correction particularly difficult to specify, and it was reevaluated during the final stages of the development of the new guidelines. Rearranging the shape of the design precipitation sequence reduced the flood peak in all basins (II).

Figure 13 shows the sensitivity of maximum water level to changes in design precipitation for the reservoirs Lossen, Sveg and Arbråsjöarna in the Ljusnan river.

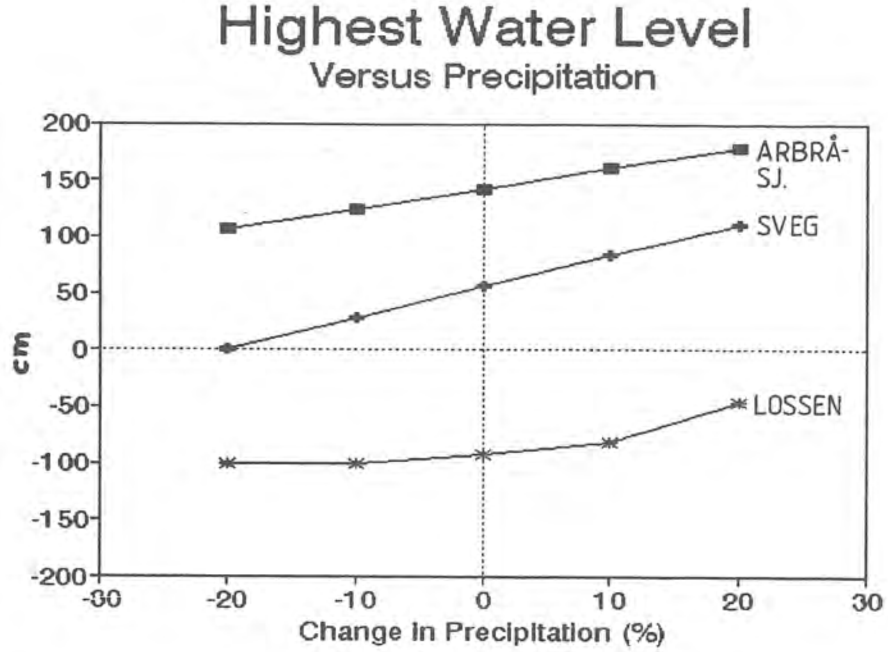

Figure 13. Effect of changes in the areal design precipitation sequence, on highest water level for three reservoirs in the Ljusnan river (from III).

5.1.2 Hydrological prescriptions

The magnitude of the snowpack had a great influence on the spring peak flow and water stage, almost as large as that of the design precipitation (Figure 14). A ten percent increase of the prescribed 30-year snowpack (corresponds approximately to a 60-year snowpack), increased resulting spring flood peaks with about 4 %. The areal distribution of the initial snowpack was less critical, but the snow culmination date was important. The snow specifications did not affect the simulation of the most critical autumn floods.

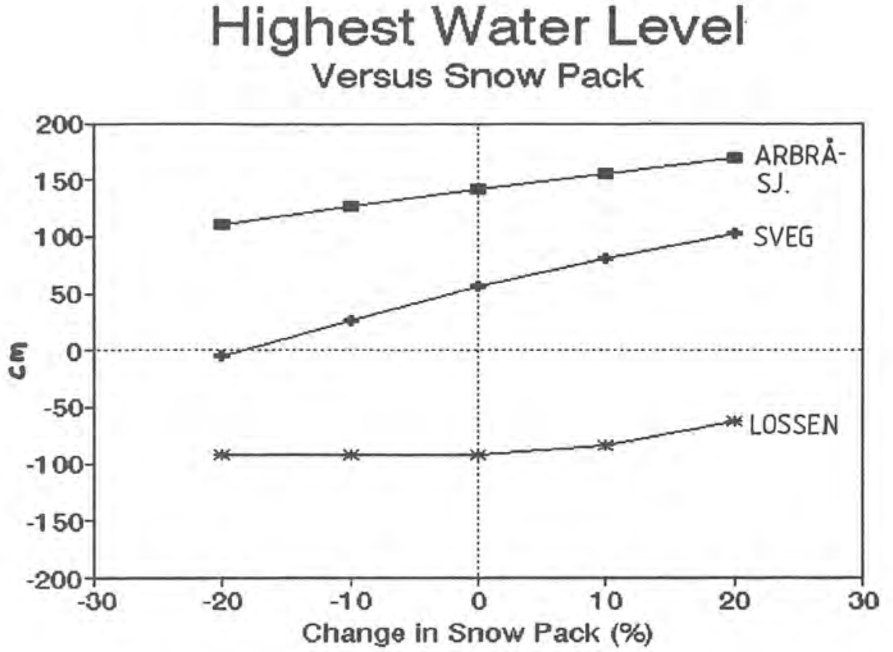

Figure 14. Effect of changes in design snowpack on maximum water level (from III).

The condition of fully saturated soil at the beginning of spring was most important for the southern basins and only for the spring flood simulation. Figure 15 illustrates the sensitivity of the spring floods to changes in the initial conditions; snow culmination date, snowpack size, snow distribution in the basin and initial soil moisture deficit.

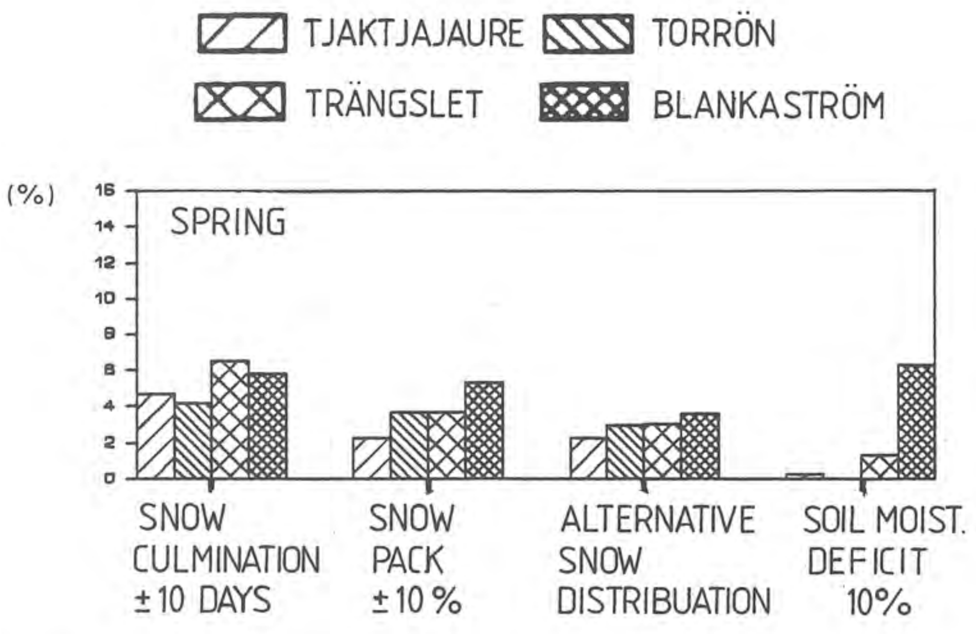

Figure 15. The largest absolute sensitivity of the most critical spring flood peaks, due to changes in the prescribed hydrological conditions (from II).

5.1.3 Modelling procedure

The iteration procedure during which the design rainfall sequence is moved in time, is important for finding the most critical timing of events. It is difficult to know in advance whether the spring or the autumn case will be most critical. For some reservoirs the most critical spring and autumn water stages were of the same magnitude. The length of the critical period of timing of the design rainfall varied greatly between the studied reservoirs. Thus, to avoid underestimation of the design flood the design rainfall sequence should be tested at all dates of these periods (Figure 16).

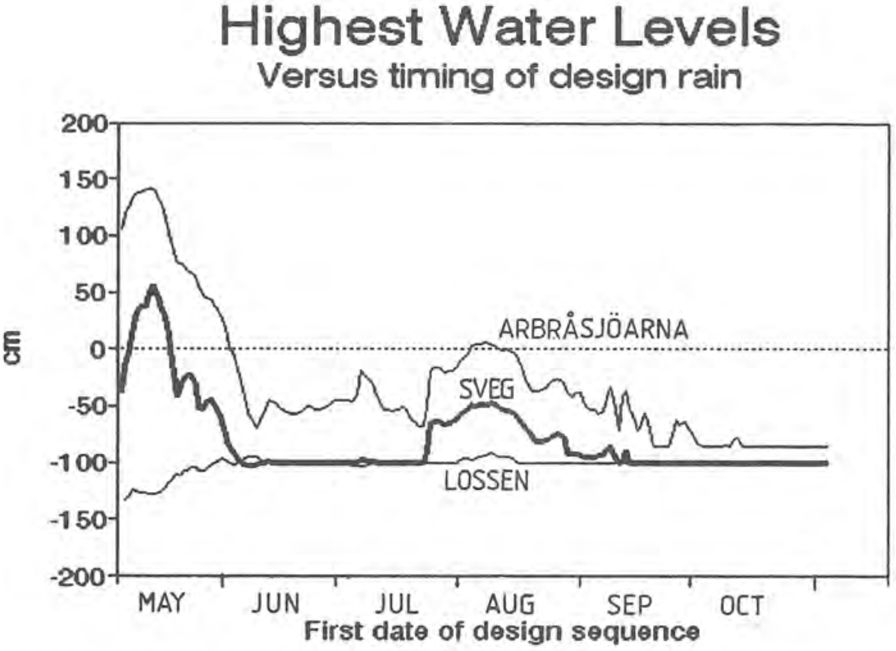

Figure 16. The highest water levels versus timing of the design rainfall sequence over a ten-year simulation period (from III).

Furthermore, the decision of Flödeskommittén (1990) to employ a conceptual hydrological model with snow and soil routines (the HBV model or similar) in the simulations was found to be essential. In Southern Sweden, where soil moisture deficits are substantial in summer, simulations with a simplified model, excluding evapotranspiration, resulted in 12 % to 85 % larger floods (II). A hydrological model is also needed for the modelling of snowmelt floods and for applications to the large river basins of the north where spring or even summer conditions may prevail in the lowlands while there still is winter in the mountains. This means that one event may result in snow accumulation in the upper parts of the basin and flood generation in the mid parts, gradually reduced by the increasing soil moisture deficits in the lower parts.

Spillway capacity was crucial when simulating reservoir water stage development (Figure 17). A slight increase in spillway capacity can improve the situation for a particular dam considerably but could at the same time make the situation worse for downstream reservoirs. The resulting flood from different timing of release from upstream dams, local inflow and reservoir operation is difficult to know in advance and the critical events will differ between reservoirs within a system and between different systems. However, the results from the sensitivity analysis of water stage development in the Ljusnan system indicate that the critical flood and water stage development increases the further down the system one gets (III).

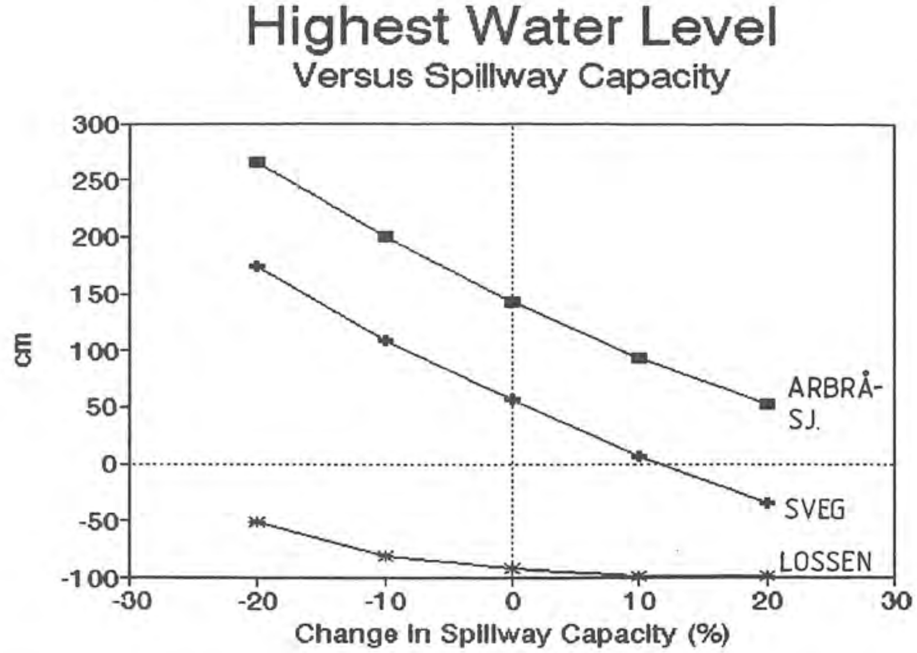

Figure 17. Effect of a change in spillway capacity on maximum water level (from III).

5.2 Model calibration and design flood simulation

The guidelines do not specify a detailed calibration procedure, they merely state that the calibration should emphasize the largest observed floods. This section gives the sensitivity and resulting uncertainty of the design floods due to model calibration.

5.2.1 Parameter sensitivity

The results from the Monte Carlo simulations (V) were grouped in three classes of parameter sensitivity; sensitive, moderately sensitive and insensitive parameters. This was done by comparing the cumulative distributions of the parameters which gave acceptable and unacceptable results. Examples of the frequency distributions for the three sensitivity classes are shown in Figure 18.

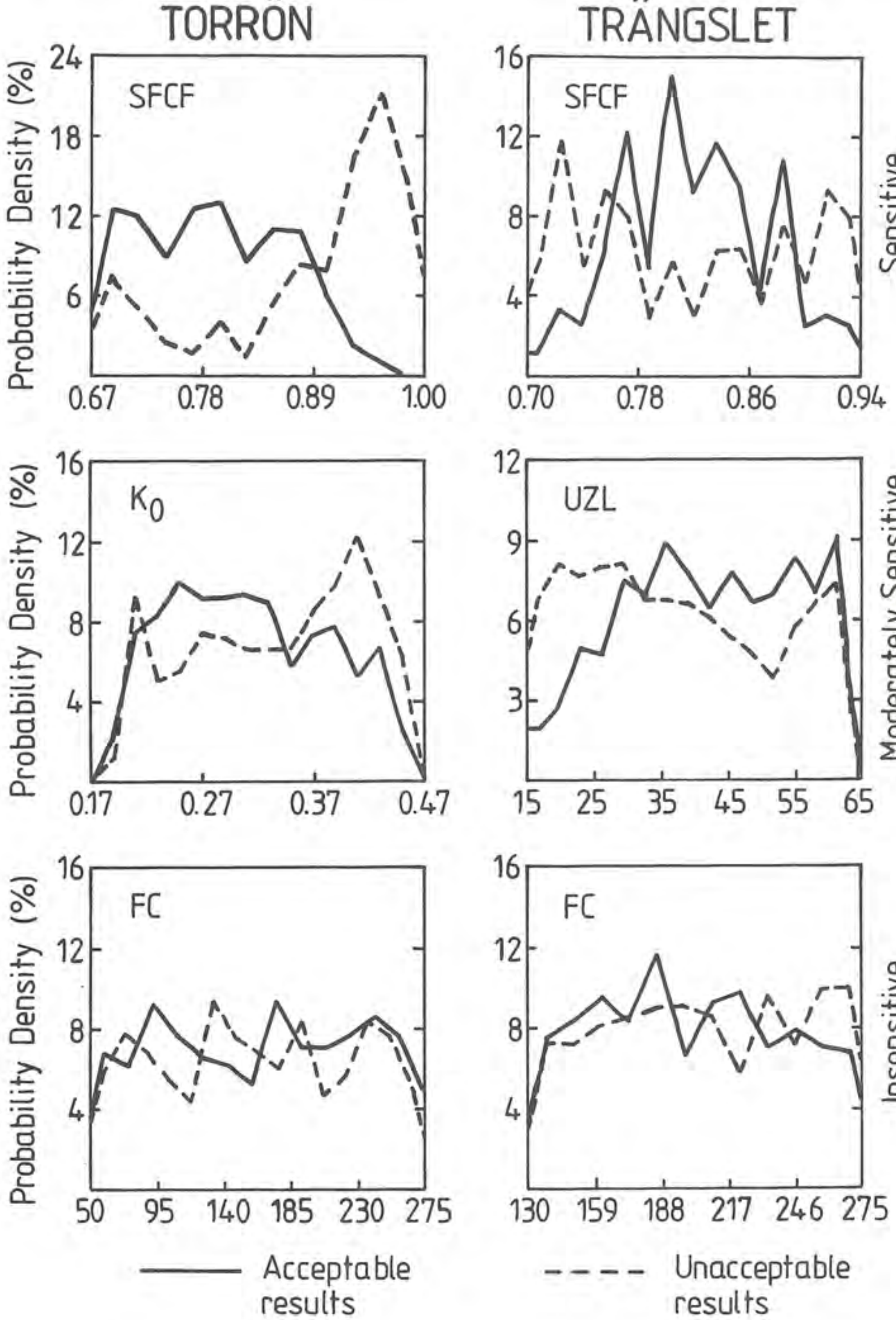

Figure 18. Examples of the parameter frequency distributions for acceptable and unacceptable results for three sensitivity classes (from V).

The simulated runoff was in general most sensitive to the snowfall correction factor (SFCF), the degree-day factor (CFMAX) and the highest recession coefficients (K_0 and K_1). However, when using the model to extrapolate extreme floods the importance of the parameters changed. Generating design floods with the HBV model requires special attention to the quickest recession parameter, K_0. The influence of this parameter increased considerably when extrapolating beyond the range of the observed floods (II, III, V).

Small catchments, with a rapid response, were also sensitive to an increase in the routing parameter MAXBAS. The design floods were less sensitive to the other model parameters (Figure 19).

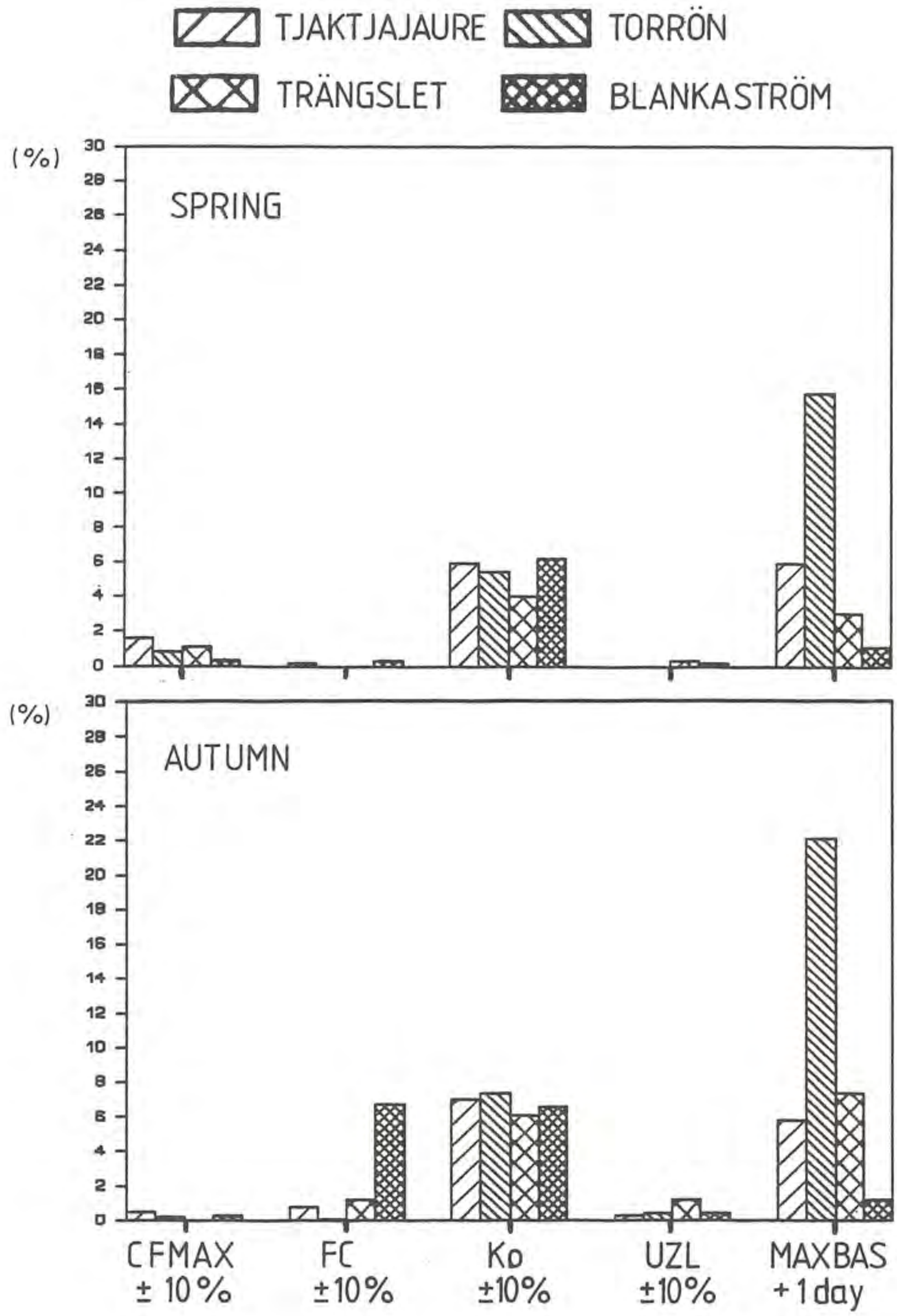

Figure 19. The sensitivity of simulated critical flood peaks to changes in the HBV model parameters (from II).

5.2.2 Parameter uncertainty and design flood simulation

One measure of parameter accuracy is the R^2 criterion of model performance, defined and discussed in Section 4.2.1. Figure 20 is an illustration of how model performance, expressed as a R^2 value, relates to maximum water stage in a design simulation. In producing this figure a volume error criterion was combined with the R^2 value for a 20 year simulation period, to form three performance levels. Level I represents a high level of accuracy, II moderate accuracy and III low accuracy.

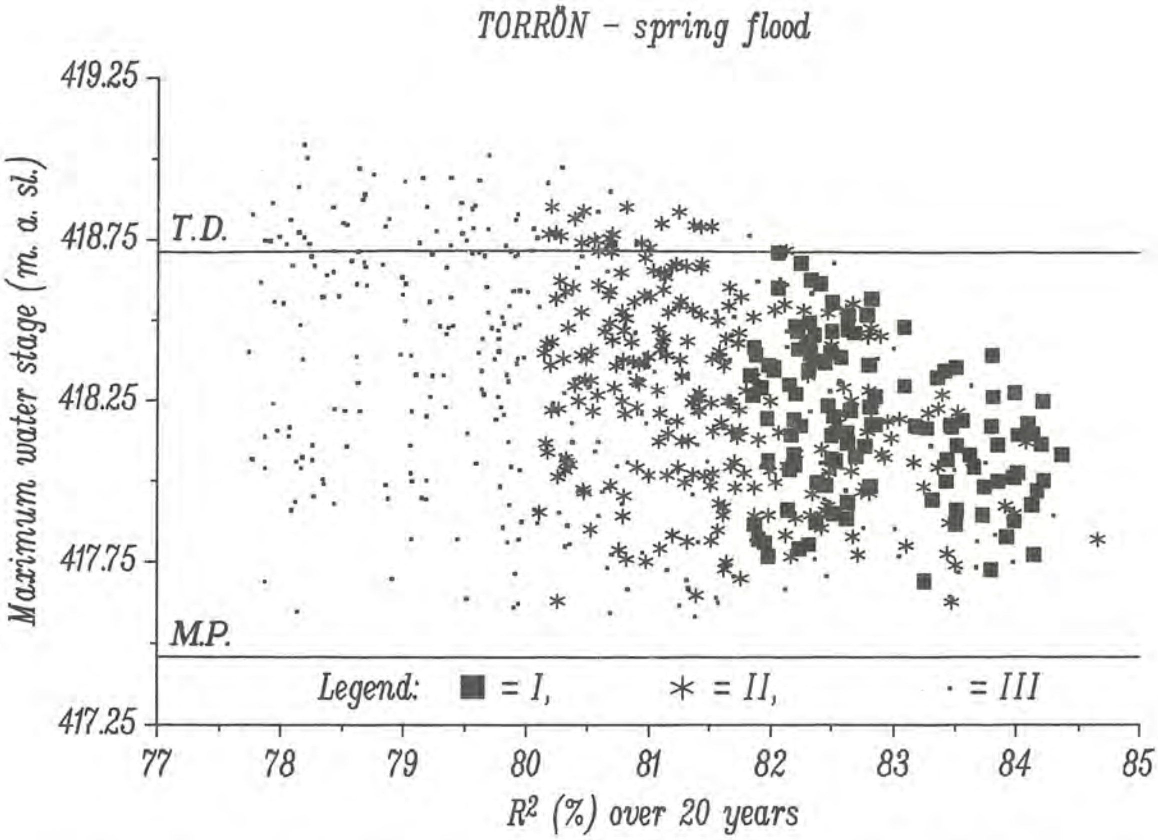

Figure 20. Maximum water stage for the most critical spring flood to the Torrön reservoir at three levels of calibration accuracy (I, II and III). T.D. = Top of dam, and M.P. = Maximum pool elevation (from V).

The variance in maximum water level increases rapidly with increasing parameter uncertainty. But, this variance can be removed by reducing the uncertainty in only a few parameters (Table 4), of which the recession parameter K_0 is the most important (Figure 21). The variance in the resulting spring floods was also due to the uncertainty in the initial 30-year snowpack. One way to reduce the effect of calibration uncertainty on the frequency analysis of initial 30-year snowpack would be to base the analysis on simulations where the yearly snowpack is updated against observed spring flood runoff. If this is done, the yearly snowpack would thus be less dependent on calibration, and the observed runoff would be utilized to a greater extent. In practical applications, however, the uncertainty in the critical parameters can not be totally removed.

Table 4. The coefficient of determination (r^2) from stepwise forward selection multiple linear regression between maximum water stage (Wst_{max}) and flood peak (Q_{max}) against the model parameters. Parameters are listed in order of selection (from V).

	Torrön		Trängslet	
	Parameters	r^2 (%)	Parameters	r^2 (%)
Q_{max}-spring	K_0 MAXBAS CFMAX	96.9	K_0 TT SFCF	95.2
Wst_{max}-spring	K_0 CFMAX SFCF	75.6	K_0 SFCF TT	90.1
Q_{max}-autumn	K_0 MAXBAS UZL	97.8	K_0 FC LP/FC	96.3
Wst_{max}-autumn	K_0 MAXBAS β	97.3	K_0 LP/FC FC	94.5

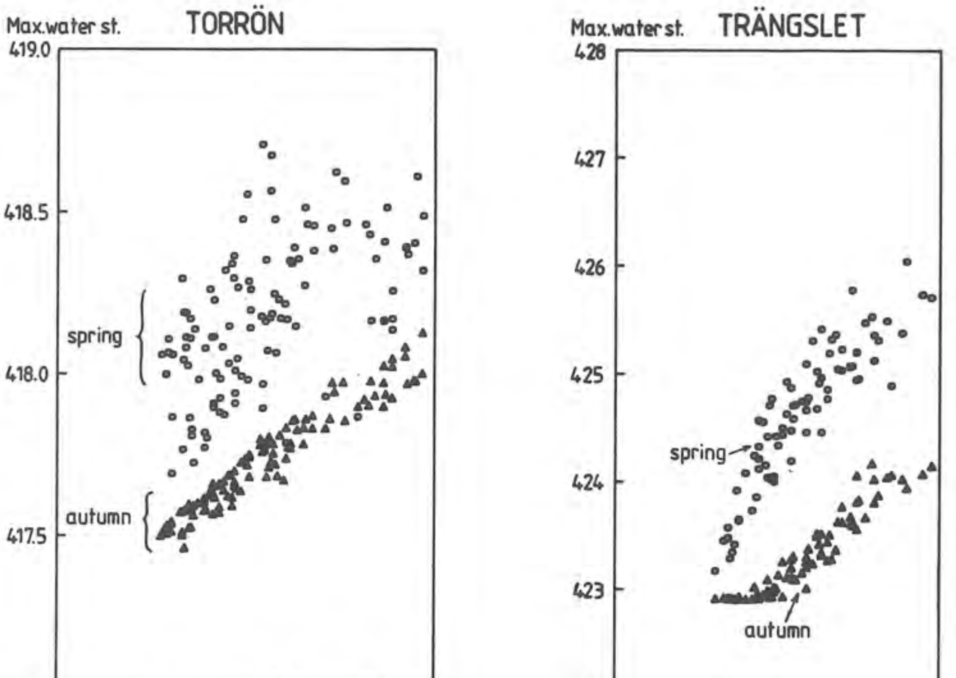

Figure 21. Maximum water stage [m.a.sl.] versus the highest recession coefficient K_0, for parameters of accuracy level I (from V).

For calibrations of performance level I, the recession parameter K_0 varied between ± 40 and 50 % for different calibration methods and different data periods in the Torrön and Trängslet catchments (V). If the calibrations with the best model performance over the largest floods were selected, the variation in K_0 was about ± 20 %. Consequently, the uncertainty due to this parameter on the design flood peak is of the order of ± 20 %. Torrön and Trängslet are catchments with good data quality and good model performance. For catchments with poor data quality, the uncertainty due to calibration on design flood simulation will be larger. The corresponding uncertainty on reservoir water stage development will depend on local conditions.

5.3 Model structure and design flood simulation

The choice of hydrological model will affect the results when applying the Swedish guidelines for design flood determination. This section focuses on catchment response during extreme floods and on how model structure influences design flood simulation.

5.3.1 Flood recession analysis

The shape of the hydrograph is a function of the catchment characteristics and the meteorologic/hydrologic inputs and outputs over time. It is normally analysed as consisting of a base flow component and a flood component. The flood runoff is the portion of the hydrograph that responds quickly and is clearly related to a given storm or snowmelt period.

The receding limb of a flood hydrograph is strongly related to storage and change in storage in the catchment after the rainfall or snowmelt stops. In general, the recession curve is expressed as:

$$Q_t = Q_0 \cdot e^{-K \cdot t} \tag{16}$$

where: Q_t = discharge at time t,
Q_0 = the discharge when t = 0, and
K = the recession coefficient.

Taking logarithms, Equation 16 becomes:

$$\ln Q_t = \ln Q_0 - K \cdot t \tag{17}$$

which can be plotted as a straight line with the gradient -K in semi-logarithmic scale. Equations 16 and 17 describe the response of a linear storage/outflow relation - a linear reservoir. The definition of the linear reservoir is

$$Q(S) = K \cdot S \tag{18}$$

This concept is often used in hydrological modelling, in particular for modelling base flow runoff. The linear reservoir is, however, normally not appropriate for modelling floods due to the fact that K is rarely constant throughout a flood recession.

Figure 22 shows rain flood recessions in semi-logarithmic scale for the catchments Äcklingen and Ljusnedal. It is evident from this figure that the slope of the curves changes with discharge magnitude, hence a linear storage/outflow relation (Equation 18) does not hold true for flood periods.

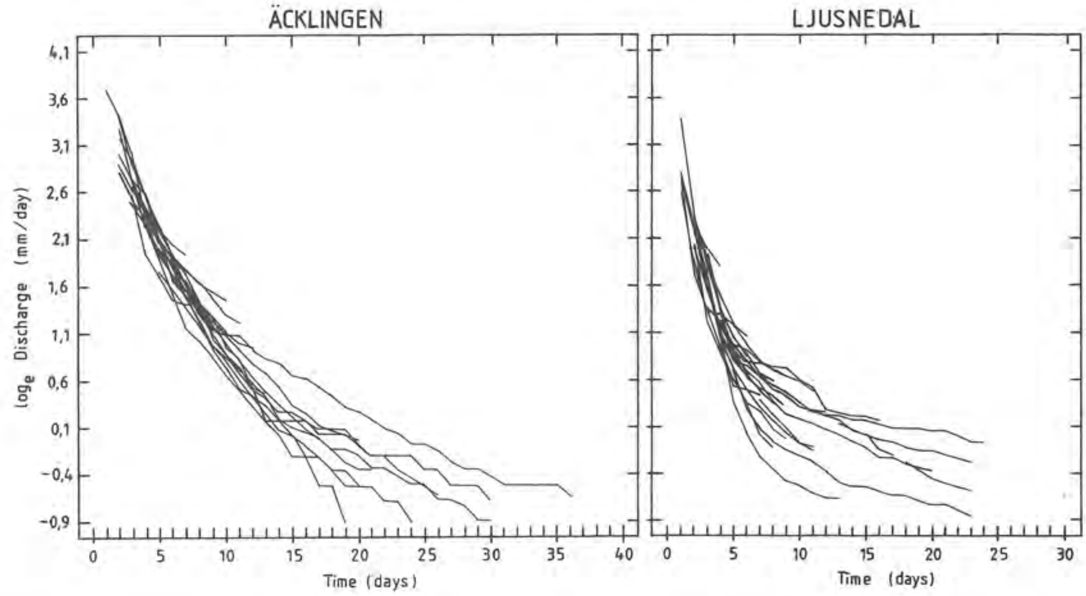

Figure 22. Recession curves for rainfloods at Äcklingen and Ljusnedal, as ln (discharge) versus time plots (from VI).

The recession curves for Äcklingen and Ljusnedal indicate that the recession rate increases with flood magnitude. Of this follows that basin response is more rapid during large floods than during normal conditions.

5.3.2 Modelling the hydrological response of extreme floods

The model component of conceptual hydrological models that describes runoff response, is called the runoff-response function. It is often taken as a non-linear reservoir formulation. In the HBV model runoff response is described by two linked tanks, where the upper tank has two outlets and will dominate the runoff generation during extreme floods (see Equations 4 and 5, and Figure 8). In Norway a simplified HBV model, equivalent to the upper response tank is used for extreme flood modelling (Andersen et al., 1983).

Experiments with several response function equations lead to two alternative formulations to the original five parameter HBV equations (VI). These were firstly a two-tank response function which was called "the E-box". The outflow from the upper response tank is given by

$$Q_u(t) = e^{K_1 \cdot S_{uz}(t)} - 1 \qquad (19)$$

where: Q_u = runoff generation from the upper response tank,
K_1 = recession coefficient,
S_{uz} = storage in the upper response tank, and
t = time.

From the upper tank water percolates with the rate PERC to a lower tank, identical with the original HBV model. The outflow is given by

$$Q_l(t) = K_2 \cdot S_{lz}(t) \tag{20}$$

where: Q_l = runoff generation from the lower response,
K_2 = recession coefficient, and
S_{lz} = storage in the lower response tank.

Consequently, the E-box has three calibration parameters: K_1, K_2 and PERC.

The second alternative response function, named "the Ln-box", was formulated as a single non-linear reservoir, with only two parameters according to

$$Q(t) = K_1 \cdot S(t)^{(1 + K_2 \cdot \ln S(t))} \tag{21}$$

where: Q = runoff generation,
K_1, K_2 = recession coefficients, and
S = storage in the response tank.

Both the E-box and the Ln-box have non-linear recession properties with fewer free parameters than the HBV formulation. Calibration and verification simulations at Äcklingen and Ljusnedal revealed that they were equally good as the HBV runoff-response function for flood simulation but poorer for baseflow simulation (VI).

In order to check the uncertainty due to model structure when modelling extreme floods, the three response functions were calibrated on small to moderate rainfloods and verified on large observed rainfloods. The reason for selecting rainfloods only, was to reduce the influence of the snow routine and thereby get a clearer picture of the runoff-response function behaviour (Figures 23 and 24). All three functions were run with the snow and soil routines of the HBV model. For most of the studied floods, however, the snow routine was not active, since rainfloods normally occur during summer and early autumn.

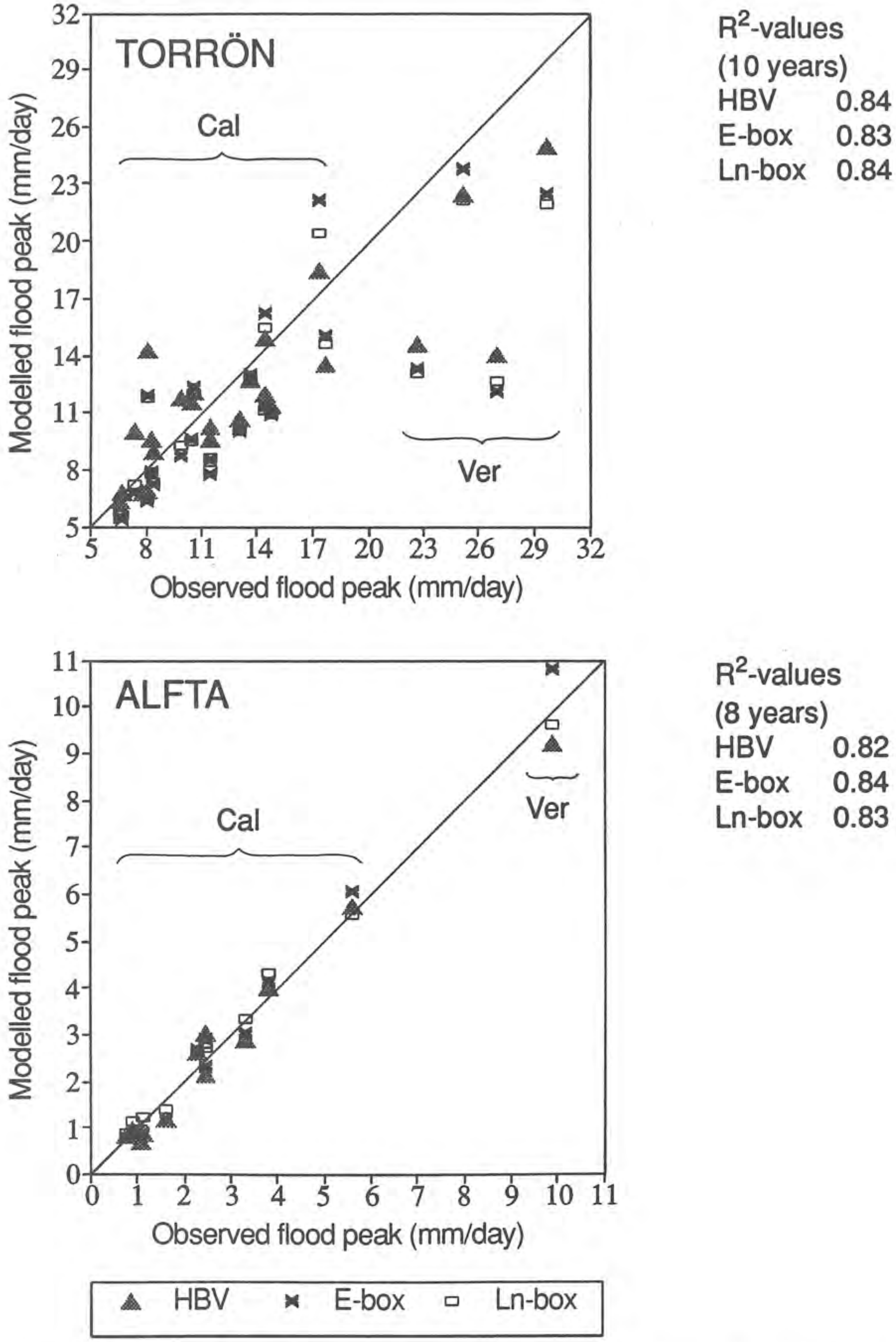

Figure 23. Model performance for three alternative runoff-response functions at Torrön and Alfta. The functions were calibrated and verified on rainfloods only. R^2 values refer to calibration periods (from VI).

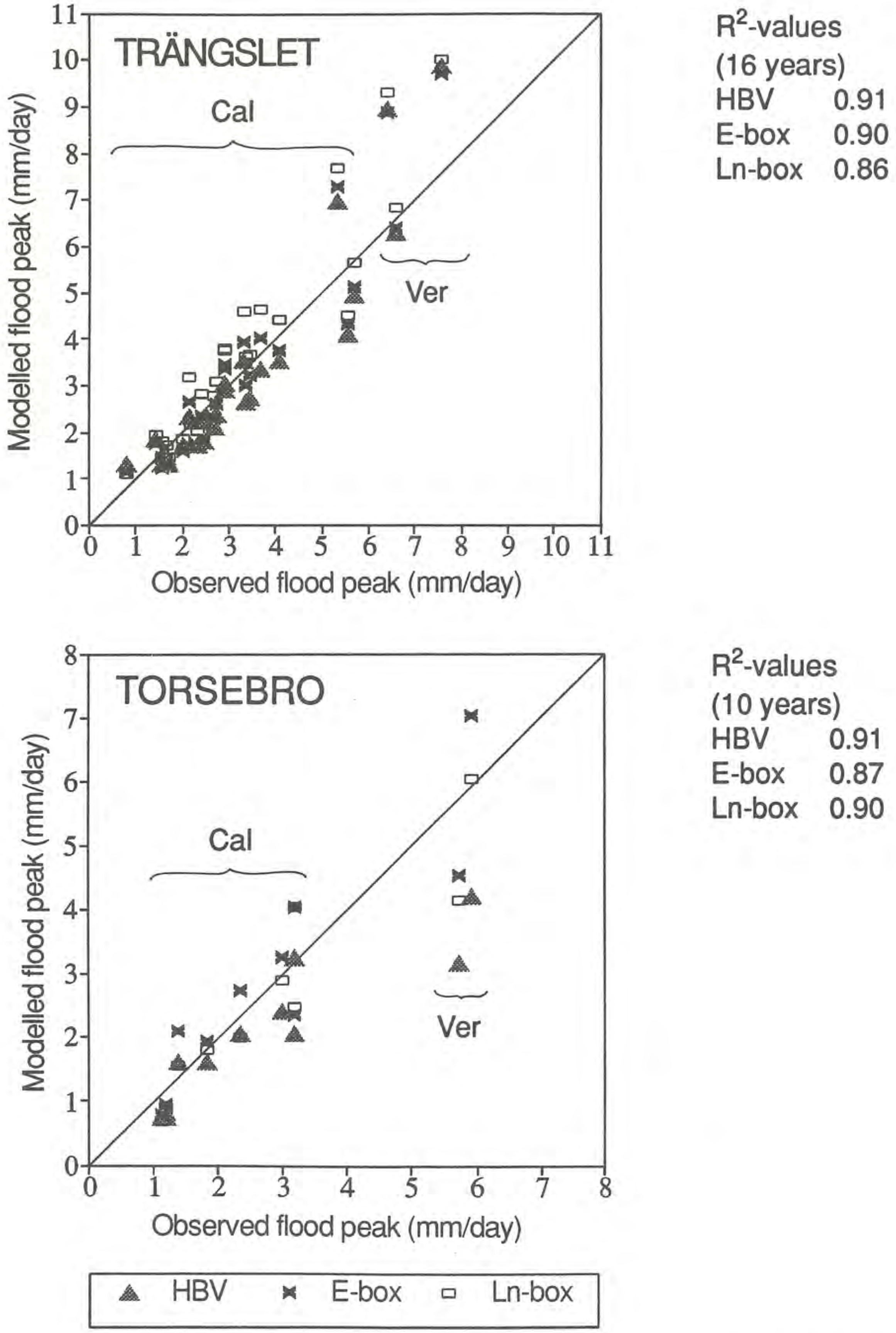

Figure 24. Model performance for three alternative runoff response functions at Trängslet and Torsebro. The functions were calibrated and verified on rainfloods only. R^2 values refer to calibration periods (from VI).

From the results shown in Figures 23 and 24 it is difficult to select any response function as performing clearly better than the others. Furthermore the figures show that the quality of the model performance over the verification periods is related to that over the calibration periods.

Much of the errors in the extreme flood modelling was due to errors in estimated areal rainfall. The models were run on weighted point measurements of precipitation. At Torrön there also occurred some snow accumulation during the largest flood, which obviously reduces runoff. Another problem with this catchment is that it has an immediate runoff response. If heavy rainfall for example starts in the evening one day and ends at midday the following day it will be divided over two time steps in the model, and the modelled flood will have a lower peak than the actual one. Design flood simulation according to the Swedish procedures avoid these problems, since all meteorological and hydrological inputs are preset on a daily time step. However, for catchments with a quick runoff response or with a small reservoir, a shorter time step may be used (Flödeskommittén, 1990).

To illustrate the effects of model structure on design flood simulation, the three response functions were recalibrated over the largest observed rainfloods and thereafter used to simulate the most critical autumn floods from design flood simulation in the test basins (Table 5).

Table 5. Design flood simulation of the most critical autumn floods using the HBV model with three different runoff-response functions. The table gives the resulting peak flow values (mm/day). The range is calculated with reference to the mean of the highest and the lowest peak values (from VI).

	Torrön (1983-10-16)	Alfta (1985-09-24)	Trängslet (1985-08-18)	Torsebro (1988-08-16)
HBV	63.3	14.2	19.6	10.1
E-box	113.1	18.5	31.8	14.3
Ln-box	91.5	16.6	25.1	13.0
Range	± 28 %	± 13 %	± 24 %	+- 17 %

From Table 5 it is seen that the uncertainty range of the design flood peak due to model structure is of the order of ± 20 %.

5.4 Probability of the design floods

It is extremely difficult to estimate the exceedance probability of PMF scale floods (U.S. Department of Commerce, 1986). This section discusses the risk of experiencing an exceedance of the design flood in relation to the lifetime of the dam and the design flood return period. Furthermore, the return periods of the Swedish design floods are discussed by comparisons with observed extreme floods and flood frequency analyses.

5.4.1 Return period and risk

What is the risk of experiencing an exceedance of the design flood during the lifetime of a dam? For each year of the lifetime of the dam, an exceedance of the design flood may either occur or not occur. If the annual exceedance probability of the design flood is P and the maximum flow of all years (N) are statistically independent, the probability that the annual maximum flow will exceed the design flood X times is given by the Binomial distribution (e.g. see Haan, 1977) as follows:

$$P_x = \binom{N}{X} \cdot P^X \cdot (1 - P)^{(N - X)} \tag{22}$$

where P_x = probability of x = X exceedances of the design flood.

Because the interest is to study the case - one exceedance or more - it is the complement event to not experiencing any exceedance of the design flood that is searched for, i.e. $1 - P_x$ where X = 0. This leads to the equation:

$$1 - P_x = 1 - \left(\frac{N!}{0!\ (N - 0)!} \cdot P^0 \cdot (1 - P)^{(N - 0)}\right) = 1 - (1 - P)^N \tag{23}$$

Table 6 gives the total risk of experiencing an exceedance of the design flood for various life spans of the dam and design flood return periods by using Equation 23.

Table 6. The risk (in percent) of experiencing an exceedance of the design flood for a dam in relation to the lifetime of a dam and design flood return period.

Return period of the design flood (years)	Lifetime of dam (years)			
	50	100	200	300
50	63.6	86.7	98.2	99.8
100	39.5	63.4	86.6	95.1
500	9.5	18.1	33.0	45.2
1 000	4.9	9.5	18.1	25.9
10 000	0.5	1.0	2.0	3.0

From Table 6 it is seen that to reduce the risk level to 1 % for a 100 year lifetime will require that the design flood should have an annual exceedance probability of 1/10 000.

5.4.2 Comparison with observed floods and frequency analysis

It is an urgent but difficult task to assess the probabilities of the Swedish design floods. A lot of effort was spent on verification and control during the development of the guidelines. First of all, computed design floods were compared to actual observations in the rivers, and secondly, comparisons were made with estimated 10 000 years floods according to standard procedures for flood frequency analysis.

Flödeskommittén (1990), Bergström (1988b) and Bergström et al. (1989) report that the highest observed floods in Swedish records, as an average, stay below 45 % of the design flood for spring conditions whereas the corresponding number for summer and autumn is 40 %. Single extreme values did not exceed 65 % of the design flood after investigation of some 2 800 station-years from all parts of Sweden. They point out the uncertainty in the results from the frequency analyses, but nevertheless conclude that the design floods according to the suggested procedures have return periods that on an average exceed 10 000 years.

5.4.3 Comparison with historical floods in the Siljan basin

At Siljan there is a more than 100 year long record of observed water levels and three extremely high historical flood marks. These are the spring floods of 1659 (166.10 m.a.sl.), 1764 (166.04 m.a.sl.) and 1860 (164.95 m.a.sl.). Figure 25 shows the annual maximum spring water stage data recorded at Siljan. The lake Siljan was regarded as a reservoir with an outflow controlled by the original rating curve, in a design flood simulation[4] (VII). Thus the resulting water levels from the simulation could be compared with observed high water marks and frequency analysis.

The most critical spring flood, simulated according to the Swedish design flood guidelines, caused Siljan to rise to an elevation of between 166.52 and 166.85 (m.a.sl.) depending on which extrapolation method of the original outlet rating curve that was used. In terms of frequency, estimated with the Gumbel frequency distribution, this corresponds to 3 450 and 6 495 year return periods if the analysis is based only on the systematic period of annual maximum water level observations, Figure 26. When the three historical floods were incorporated in the frequency analysis the return periods dropped to 590 and 965 years respectively, Figure 27.

[4] This project was carried out during the development of the Swedish guidelines for design flood determination. The final guidelines differ slightly from the ones applied at Siljan, mainly in that the altitude correction of the design rainfall sequence is reduced and that both spring and autumn floods are computed with a hydrological model including soil moisture routines.

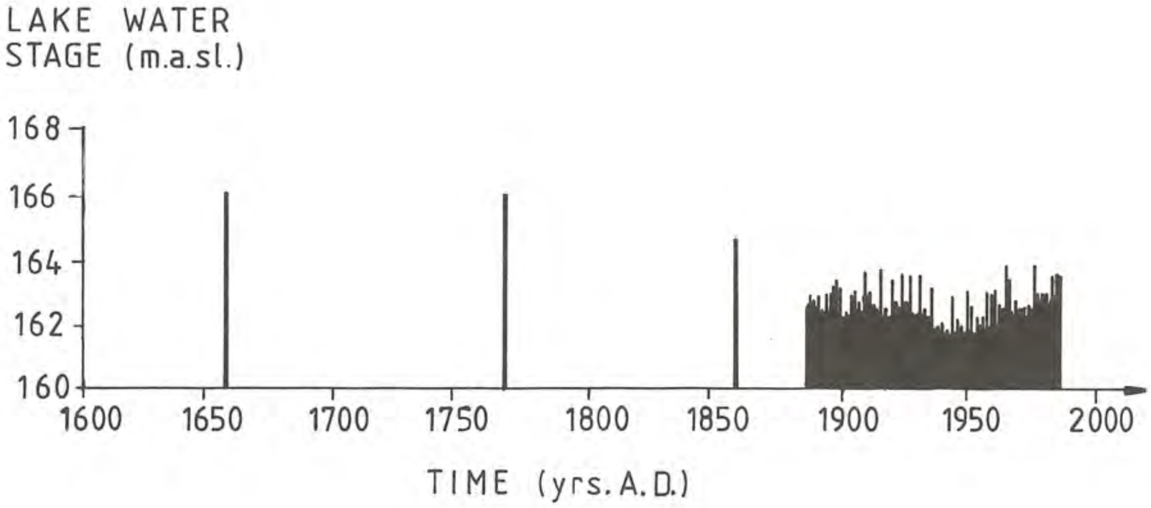

Figure 25. Annual maximum spring water stage data at the lake Siljan (from VII).

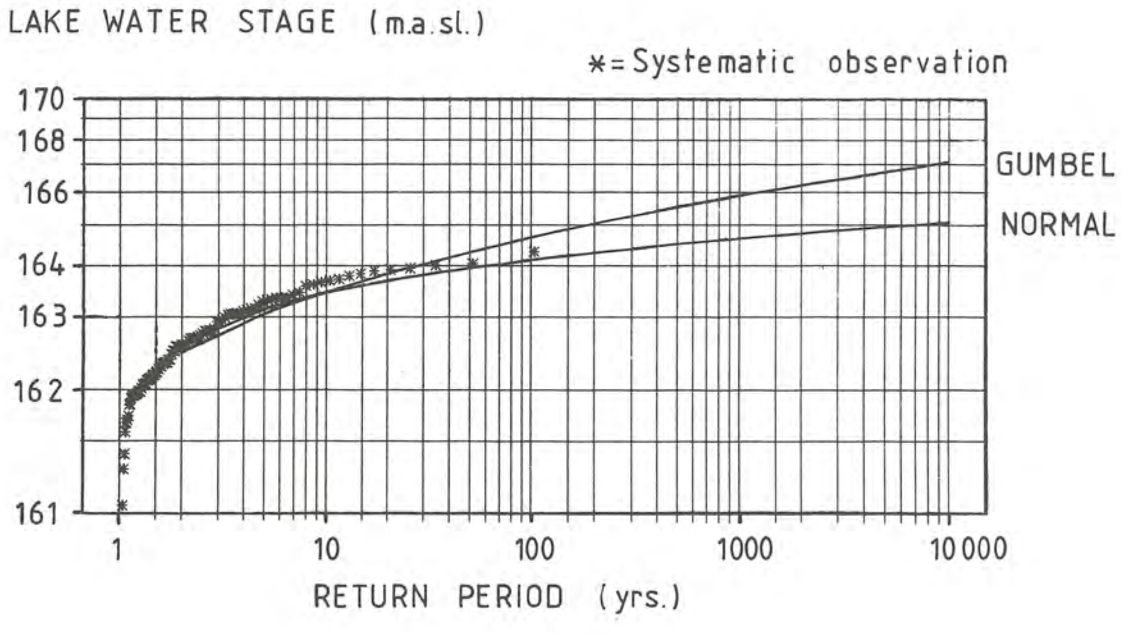

Figure 26. Frequency analysis based on systematic recordings of annual maximum spring water levels. The Gumbel distribution was used and for reference the normal distribution is depicted (from VII).

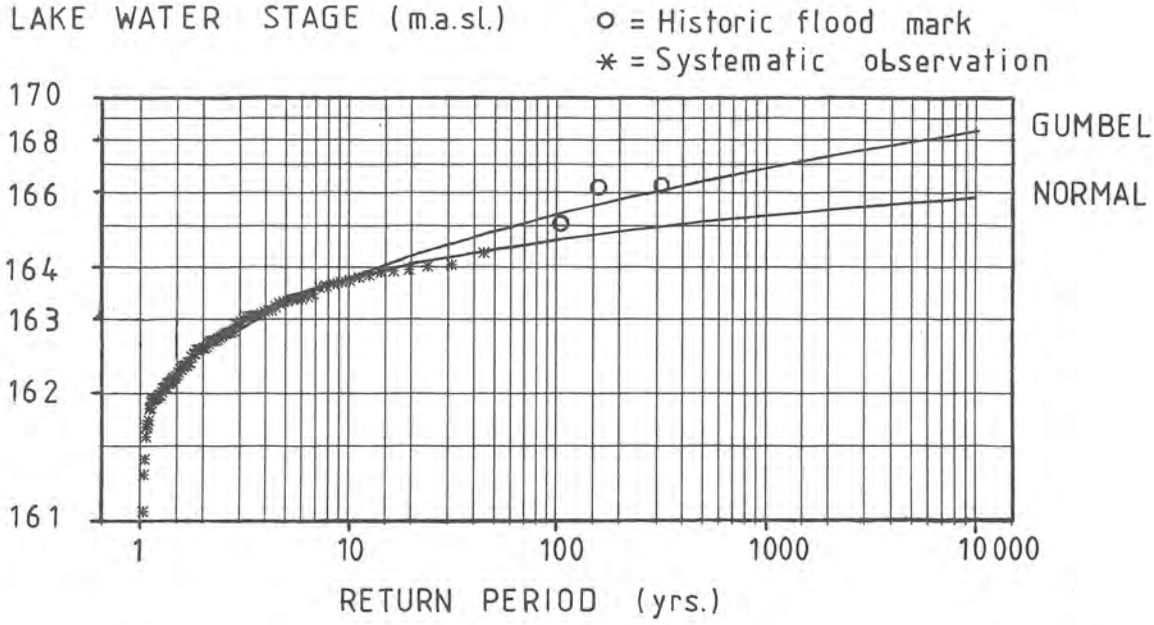

Figure 27. Frequency analysis based on systematic recordings of annual spring maximum water levels and historical flood marks. The Gumbel distribution was used and for reference the normal distribution is depicted (from VII).

Elevations of 164.54 or 164.58 (m.a.sl.) were the result of routing the most critical autumn flood through Siljan. No historical flood marks for autumn floods have been made. Frequency analysis using the Gumbel distribution resulted in return periods for the most critical autumn flood of between 1 180 and 1 280 years.

It was found that the most critical spring design flood simulation yielded water levels of about half a meter above the extreme historical flood marks. If such a hydrologic situation would occur it would result in an increase of the Siljan water level, disregarding the effects of regulation, of about 6 meters. A normal year the spring flood would lift the lake about 2 meters.

The return periods of the design floods were lower than first expected. The return period for both the spring and the autumn design flood was estimated to about 1000 years opposed to beyond 10 000 years, as was concluded by Bergström et. al. (1989). This could possibly depend on that their analysis mainly concerned streamflow, while this study dealt with water levels of a lake with a large damping effect on inflow, and a long hydrological memory.

However, the frequency analysis of water stage development at Siljan was very sensitive to choice of frequency distribution and used data. Return period differences between frequency distributions was in the order of ten to three hundred times. The second largest source of uncertainty was if the historical flood marks were incorporated or not. When the historical flood marks were utilized, the return periods dropped about ten times. Since it is always questionable how high the historical floods were and if there has been any change to the outlet of the lake over time, the results from frequency analysis incorporating historical flood information have to be interpreted with caution (VII).

5.5 Climatic variability

Climatic variability and climate change are debated issues at present. This section discusses the effects of climatic variability on hydrological modelling of observed as well as design floods.

5.5.1 Consequences for hydrological modelling

Calibration of the HBV model requires a runoff record covering a number of hydrological events. In Swedish catchments large rain floods are rare but needed for calibration of recession parameters. A glance at the runoff records from the major Swedish rivers indicates that the 60-ties represent average conditions, the 70-ties dry and the 80-ties wet hydrological conditions. In paper IV, the effect of calibrating the model on wet, average and dry periods was studied. It was found that the effects of climatic variability leveled out quickly and only between two and six years were needed to fit the model parameters accurately (Figure 28).

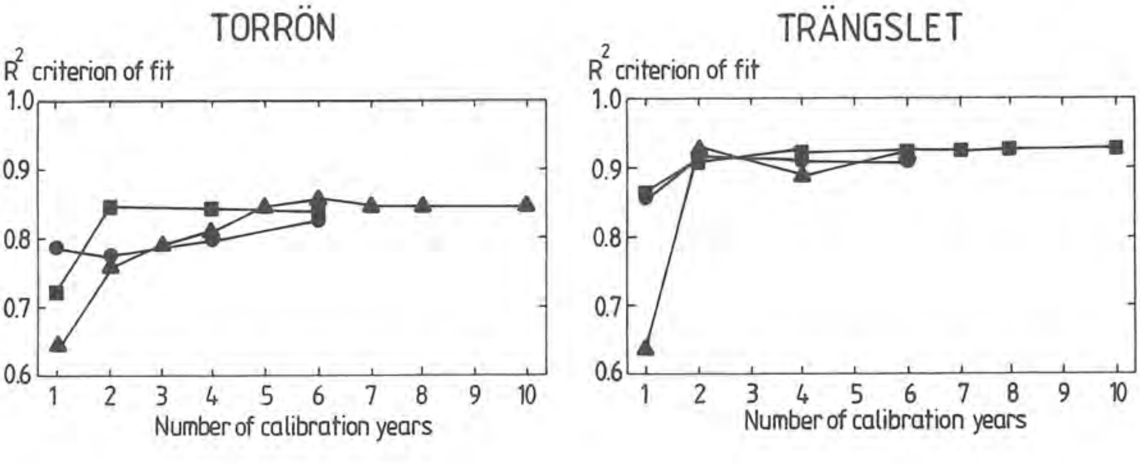

Figure 28. Model performance after POC, at Torrön (20 years) and Trängslet (18 years), for alternative lengths of the calibration period. Squares depict initially wet, circles initially average and triangles initially dry calibration periods (from IV).

With a calibrated model short term effects of climate variability on runoff can be modelled. Many researchers are currently also estimating the effects of a changed climate by hydrological modelling. The most frequently used climate change scenario is that resulting from a doubling of the carbon dioxide content in the atmosphere, which is expected to occur around year 2030. Climate change estimates based on simulations with a general atmospheric circulation model (GCM) indicate that for Sweden, annual mean temperature will increase with about 2.0 - 5.0 °C and annual precipitation will increase totally about 200 mm (Bach, 1989). Lettenmaier and Gan (1990) in California, Panagoulia (1991) in Greece, Saelthun et al. (1990) in Norway and Vehviläinen and Lohvansuu (1991) in Finland all show that the mentioned climate change scenario will lead to shorter winters, with less snow and more runoff and that increased evaporation in summer will reduce summer runoff. However, although the seasonal pattern of runoff will change they report that the annual change in simulated runoff volume will be small. Lettenmaier and Gan (1990) furthermore conclude that the annual flood maxima would increase. This was primarily due to an increase in rain-on-snow events, with the time of occurrence of many large floods shifting from spring to winter.

5.5.2 Consequences for design flood determination

It is difficult to estimate the consequences of a changed climate on runoff and even more difficult to estimate the consequences for extreme events such as storms and large floods. This was recognized by the Swedish Committee for Design Flood Determination, who concluded that existing scenarios of climate change are too uncertain to justify any additional safety margin (Flödeskommittén, 1990).

However, if the design flood simulation is based on different periods of observations, the effects of climate variability can be studied. The Swedish guidelines for design flood determination recommend that a record length of at least 10 years shall be used. Figure 29 shows the largest difference between a reference simulation (10 years) and simulations based on four different five year periods and one twenty year period. The twenty year simulation shows that the uncertainty due to chosen climate period for the design flood calculation was at the most 9 %.

The simulations of design floods with different degrees of parameter accuracy as discussed in Section 5.2.2 also illustrate the effects of climate variability on design flood determination. Parameter ranges were mainly compiled from parameter sets of model calibrations over consecutive 4-year periods at the Torrön catchment and 3-year periods at the Trängslet catchment. These calibrations reflect different climate periods and the resulting variance in design flood peak corresponds to the results in Figure 21 (V).

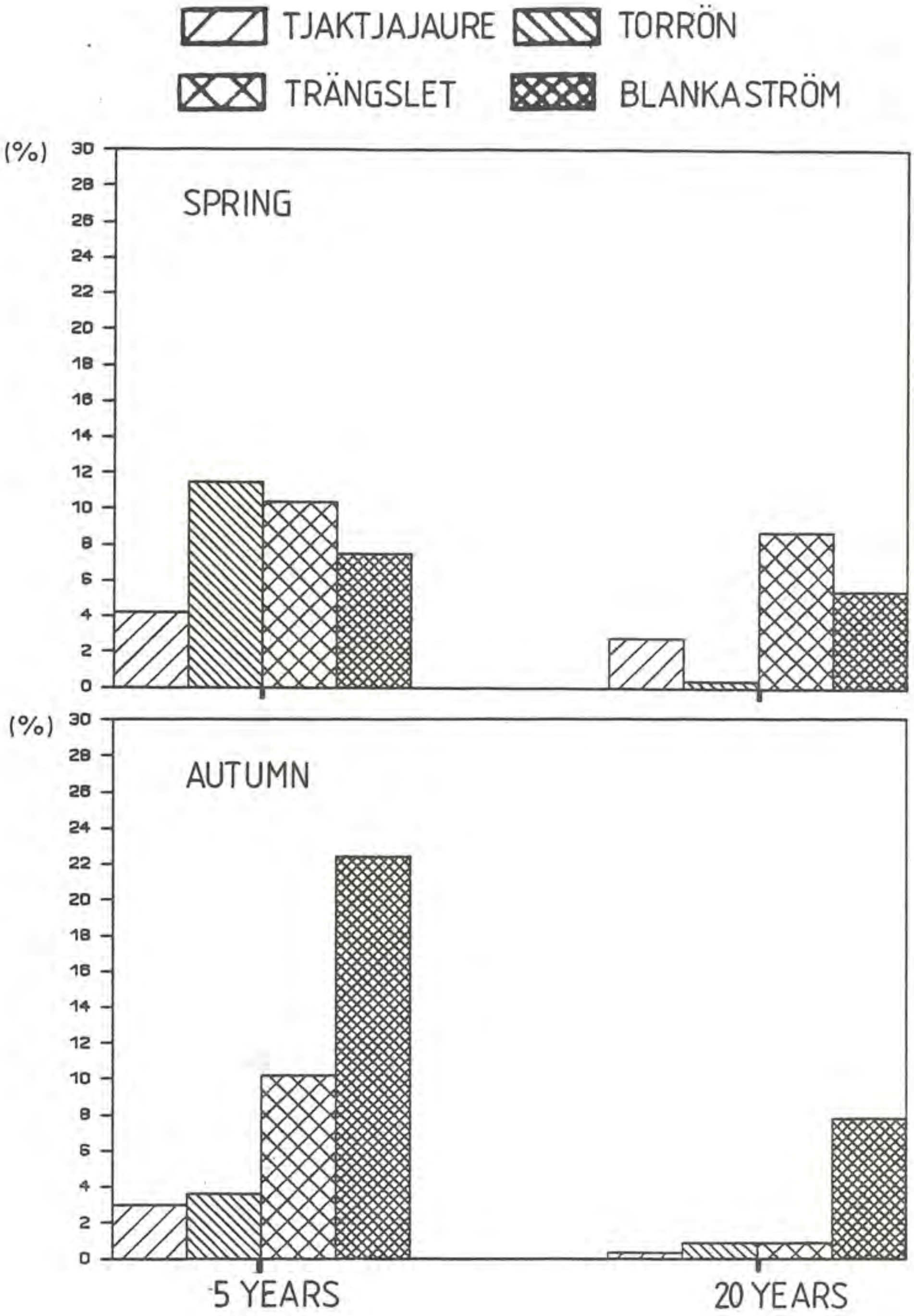

Figure 29. ***The largest absolute sensitivity of design flood peaks with respect to the climate period used. 5 YEARS refer to the largest difference between any of four five-year simulations and the reference simulation. 20 YEARS is the difference between a twenty-year and the reference simulation (from II).***

6. CONCLUSIONS

From the research reported in this thesis, the following conclusions are drawn:

The design precipitation and snow pack magnitude are the most important prescriptions in the Swedish guidelines for design flood determination. Alterations of the rainfall depth gives an effect of approximately 1:1 on autumn floods and an effect of about 1:0.6 on spring floods. Consequently, the regional design precipitation sequence and all suggested correction factors to the sequence (area, altitude and season) are crucial.

It is difficult to assess a general figure of the accuracy in the computed design flood and critical water stage development for a dam. The uncertainty in application is mainly due to model structure, calibration and selected climate period.

The uncertainty range of the design flood peak due to model structure is of the order of ± 20 %. The same order of uncertainty stemmed from uncertainty in model calibration at Torrön and Trängslet. These catchments have good data quality and good model performance. For catchments with poor data quality, the uncertainty due to calibration on design flood simulation will be larger. Uncertainty due to chosen climate period for the design flood calculation is at the most about 10 %. The simulation should therefore be based on a longer period than the suggested 10 years to reduce the influence of climatic variability on the results. The corresponding uncertainty on reservoir water stage development will depend on local conditions.

Design flood simulation with different formulations of the model structure shows that the original HBV model generates the lowest flood peaks of the compared models. However, verification runs over observed extreme floods do not reveal that the HBV model systematically underestimates extreme floods beyond the range of calibration. The HBV model is also easiest to calibrate and gives the best model performance (compared with observed runoff) of the compared models.

Most of the uncertainty caused by calibration stem from only a few model parameters in the snow and runoff-response routines. The influence of the most important parameter, the recession parameter K_0, increases considerably when extrapolating beyond the range of the observed floods. To reduce the uncertainty in this parameter, the calibration must emphasize the recessions of the largest floods. Preferably, large rain floods should be used, since the recessions then are undisturbed by melting snow.

The developed automatic calibration scheme yielded as good model performance as the manually calibrated models used for operational flood forecasting at SMHI. Automatic calibration is appropriate for simulation of experienced floods and for sensitivity analyses but is not optimal for simulating design floods. This is due to the fact that automatic calibration seeks to even out model errors over several flood components. For design flood simulations, automatically fitted parameters should be tuned over observed extreme floods separately and visually inspected.

The design flood modelling for the Ljusnan river shows that it is difficult to assess the integrated effects of extreme precipitation, snowmelt, soil moisture status and reservoir operation in a system in advance. For some reservoirs the spring flood was most critical, while the autumn flood is more severe for others. For the reservoirs within the Ljusnan hydropower system, the flood generated by simulating the total upstream area is more severe than that from simulating the local catchment only. Furthermore, the results indicate that the critical flood and water stage development increases the further down a hydropower developed system one gets.

The water stage development resulting from an extreme inflow hydrograph is influenced by the reservoir operation strategy. The regulation strategies reported here are only examples of possible ways of operating the spillways. In practical applications, the operation strategy has to be worked out in close cooperation with the River Regulation Enterprise of the studied river.

Although the design water stages are sensitive to changes in some of the design factors, these factors can often be varied considerably without changing the conclusion on whether a particular dam can meet the requirements of the new guidelines or not. There will, however, also be simulations close to the limit which deserve careful consideration.

The estimation of the annual exceedance probability of the design floods is a difficult task, which requires further research. The frequency analysis incorporating historical floods at the lake Siljan indicates an annual risk level of about 1/1000, which is much larger than the risk levels estimated by Bergström et al. (1989) and Flödeskommittén (1990). The uncertainty of the historical floods should, however, be borne in mind.

Further research is needed in hydrological model development to better understand and describe catchment response during extreme flood situations. Since extreme floods in Sweden often are generated by combinations of heavy rainfall, intense snowmelt and saturated soils, further research on the severity and interaction of these factors is also necessary.

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Norrköping, March 1992

Joakim Harlin

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Spillway design floods in Sweden, Part 1: New guidelines

by

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SPILLWAY DESIGN FLOODS IN SWEDEN

Part 1. New guidelines

by

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ABSTRACT

The new Swedish guidelines for calculation of design floods for dams and spillways are presented with emphasis on high-hazard dams. The method is based on a set of regional design precipitation sequences, rescaled for basin area, season and elevation above sea level, and a full hydrological model. A reservoir operation strategy is also a fundamental component of the guidelines. The most critical combination of flood generating factors is searched by systematically inserting the design precipitation sequence into a ten year climatological record, where the initial snowpack has been replaced by a statistical 30-year snowpack. The new guidelines are applicable to single reservoir systems as well as more complex hydro-electric schemes and cover snow melt floods, rain floods or combinations of the two. In order to study the probabilities of the computed floods and to avoid regional inconsistencies, extensive comparisons with observed floods and frequency analyses have been carried out.

DÉBITS DE CRUE DE PROJET POUR LES DÉVERSOIRS EN SUÈDE

Première partie: Description des nouvelles directives

RÉSUMÉ

Les nouvelles directives suédoises pour le calcul des débits de crue de projet pour les barrages et les barrages et les déversoirs sont présentées en insistant sur ce qui concerne les barrages à haut-risques. La méthode est basée sur un ensemble de séries de données de précipitations pour une région, mises à l'échelle pour l'aire du bassin, la saison, l'altitude et pour tout le modèle hydrologique. Une stratégie d'exploitation du réservoir est également une composante fondamentale de directives. La combinaison la plus critique des facteurs générants le débit est recherchée en insérant systématiquement une série de précipitation de projet dans un enregistrement climatologique de 10 années, où la couche de neige initiale a été remplacé par une couche de neige statistique sur 30 ans. Les nouvelles directives sont applicables pour les systèmes à réservoir unique ainsi que pour des projets hydro électriques plus complexes, et ce pour les débits provenants de la fonte des neiges, pour les débits de pluies ou pour la combinaison des deux. Afin d'étudier la probabilité de débits calculés et d'éviter les irrégularités régionales, des comparaisons extensives avec des débits observés et des analyses de fréquence ont été effectuées.

INTRODUCTION

Hydroelectric power covers some 50 % of the Swedish demand for electricity. Most of this power is generated in the large rivers of the North where the climate is characterized by long winters with substantial snow accumulation. A complex system of reservoirs is therefore needed to store water from snowmelt in spring and rain in summer and autumn. The design floods for the spillways of these dams are in focus since a flood in late 1983 in Rivers Indalsälven and Ångermanälven. A preliminary investigation indicated problems with the existing praxis for estimation of the design flood and resulted in the establishment of The Swedish Committee on Spillway Design (Flödeskommittén), which started its work in early 1985. The members of the committee represented both the hydroelectric power industry and governmental agencies (the Swedish Meteorological and Hydrological Institute). In September the same year the problem was underlined once again when an extreme rainflood in combination with a jammed gate caused the failure of the Noppikoski dam in a tributary to River Dalälven in Central Sweden.

It was the ambition of the committee to present new guidelines for the estimation of design floods for dams and spillways that meet the following requirements:

1. They shall result in a safety that is considered satisfactory and reasonable by the responsible authorities and the dam owners.
2. They shall be clear and consistent and leave as little as possible to subjective considerations.
3. They shall result in the same degree of safety in all climatological and hydrological regions of the country.
4. They shall cover rainfloods, snowmelt floods or combinations of the two.
5. They shall be applicable to single reservoirs as well as to larger systems with several regulation reservoirs.

The committee worked openly to take advantage of available national and international experience within the hydropower industry and elsewhere. Progress reports were presented on several occasions (see for example Bergström & Ohlsson, 1988; Bergström, 1988; and Bergström et al, 1989). The final guidelines, which were presented in the summer of 1990 (Flödeskommittén, 1990), differ somewhat from what has been presented in these progress reports. The most important amendment is that a full hydrological model is now required both for snowmelt and rainflood conditions in contrast to a simplified model and a separation of these two conditions as was earlier suggested.

The Swedish guidelines shall be regarded as recommendations. They have, however, reached acceptance by the hydroelectric industry and the Swedish Meteorological and Hydrological Institute, which is the national agency responsible for supervision of hydrological conditions in the country and will therefore have great impact on design studies for years to come.

The Swedish Committee on Spillway Design suggests a classification of dams into two classes, depending on the consequences of a failure. Low-hazard dams, opposed to high-hazard dams, are designed by flood frequency analysis. For these dams, the committee suggests that a return period of 100 years should be used. The following presentation is limited to a description of the guidelines for high-hazard dams, where the consequences of a failure would be dramatic and human lives would be at risk. In two companion papers (Harlin & Lindström, 1992, and Lindström & Harlin, 1992) the sensitivity of the guidelines to some factors and assumptions is analysed for single reservoir systems and multiple reservoir systems respectively.

METHODOLOGY

The committee recognized that there are no internationally accepted standards for design flood estimation. A flora of methods has developed in different countries and sometimes there is even a variety of methods within a single country. It is also clear that the record of dam incidents related to inadequate spillways is a matter of major concern for engineers and hydrologists all over the world (see, for example, ICOLD, 1988).

There seem to be two main contrasting principles in estimating design floods. One is the statistical approach based on frequency analysis (see, for example, USWRC, 1982), the other is the calculation of the probable maximum flood by climatological and hydrological considerations and models (see, for example, NERC, 1975; NRC, 1985; or NVE, 1986). The committee considered these two approaches and found that the latter is the most feasible for Swedish hydrological conditions and the configuration of the Swedish hydroelectric power production system as concerns high-hazard dams. The uncertainty introduced when extrapolating short records to low annual probabilities of exceedence, and the inability to use the method in a complex system of reservoirs, were the main arguments against the use of frequency analysis.

A closer look at international praxis left the impression that much effort has been spent on the assessment of the probable maximum precipitation, whereas less attention is paid to other flood generating factors, such as snow melt, soil moisture deficits and reservoir operation. The new Swedish guidelines differ in this respect. They are characterized by the use of a full hydrological model including a reservoir operation strategy and a critical timing of flood generating factors, which have all been experienced although not at the same time or place. The result is that very low probabilities can be reached without extrapolation of any single flood generating factor. In order to meet the requirement of objectivity (No 2. above) these factors had to be very clearly specified in the guidelines.

Climatological considerations

The work of the committee started with a nationwide investigation of extreme areal precipitation in Sweden (Vedin & Eriksson, 1988; Figure 1). Emphasis was put on the analysis of extreme areal precipitation to avoid the vague interpretation of areal reduction factors in combination with rainstorms of different origin. Thus the influence of local convective precipitation with high intensity but small areal coverage was minimized. The analysis was very labour-intensive and covered every single observation of precipitation in Swedish official records between 1881 and 1988, with emphasis on areas of 1 000 km^2 and 10 000 km^2 respectively. It was restricted to observations over 24 hours. These are the data normally available, and because of the size of the basins which feed the reservoirs, a higher resolution in time was not considered necessary. The highest observation in record was a rainstorm of 150 mm per 24 hours over an area of 1 000 km^2 in Southern Sweden in August, 1945.

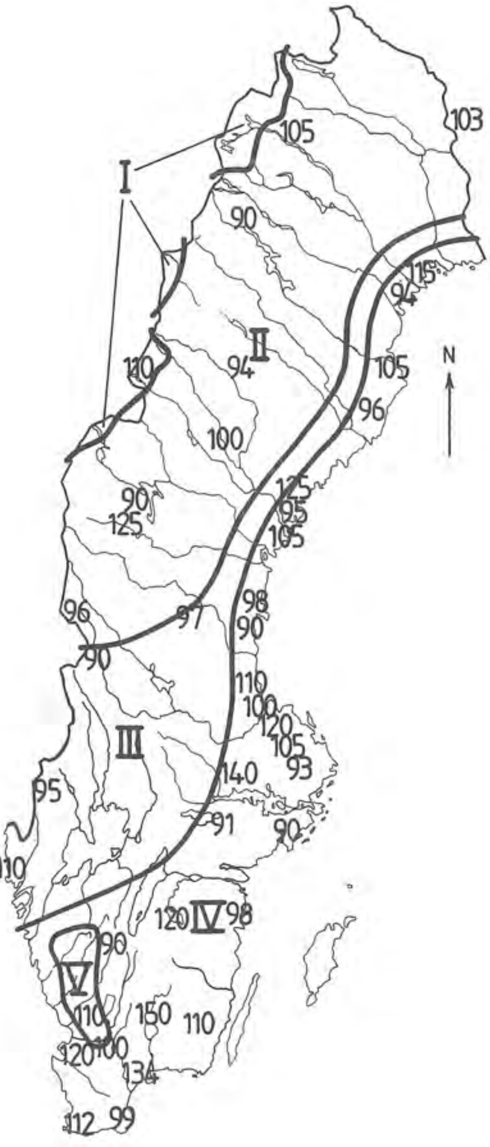

Figure 1. *The five climatological regions for choice of design precipitation sequence together with the observed maximum 24 hour precipitation (in mm) in Sweden between 1926 and 1988 over an area of 1000 km^2 (after Flödeskommittén, 1990, and Vedin and Eriksson, 1988).*

The nationwide analysis of areal precipitation included a study on intensity-duration relationships and is the foundation for five basic 14-days design precipitation sequences which are specific for each one of five climatological regions (Vedin, 1990; Brandt et al, 1987; Figure 2). These sequences were constructed so that they roughly contain the largest experienced amounts over 1 - 14 days periods. They are subject to corrections for season (Figure 3), basin area (Figure 4) and basin elevation (Table 1) before being entered into the hydrological model. It was found that the rescaling of precipitation according to elevation had to be specific for each one of the main rivers due to local effects. One argument for this differentiation is differences in proximity to the precipitation maximum close to the Norwegian coast.

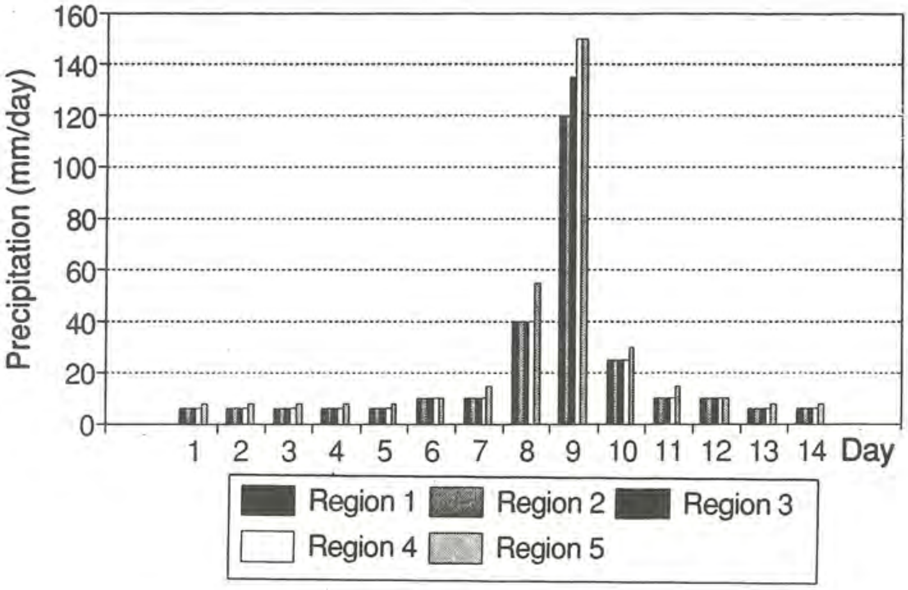

Figure 2. *14 days design precipitation sequences for the five climatological regions in Sweden. The values are valid for a 1000 km² large area below the reference level for altitude corrections and without seasonal corrections.*

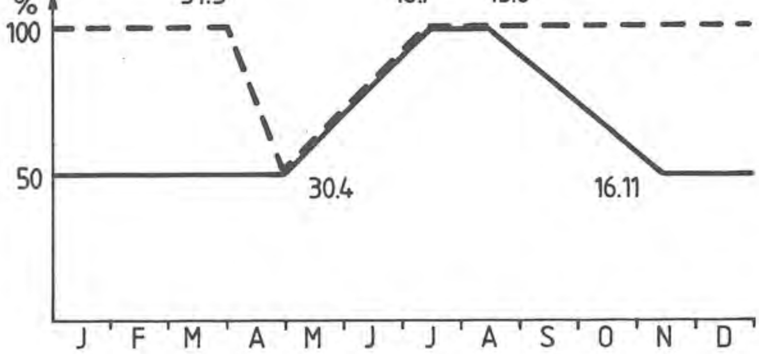

Figure 3. *Examples of seasonal correction of the design precipitation. Region 1: dashed line, regions 2 - 4: continuous line.*

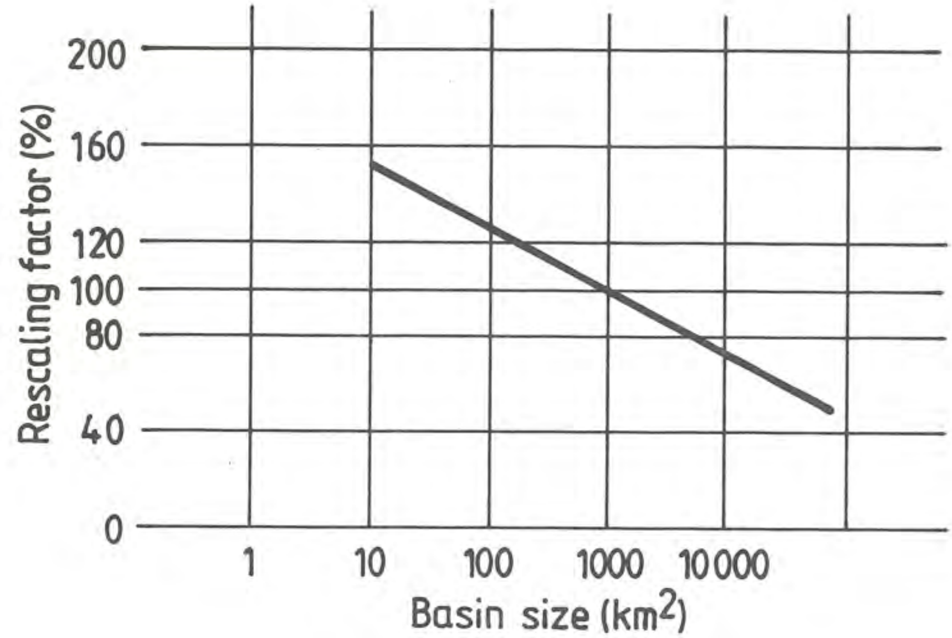

Figure 4. *Rescaling of the initial design precipitation sequences according to the size of the basin.*

Table 1. *Rescaling of the initial design precipitation sequence according to mean basin elevation above a reference altitude.*

River basin from north to south	Rescaling in % per 100 meters	Reference altitude (m.a.s.l.)
Torneälven to Indalsälven	+ 10	500
Ljungan and Ljusnan	+ 10	600
Dalälven	+ 5	600
Klarälven	+ 5	700

Hydrological considerations

The committee initiated hydrological research on the main flood generating factors and their interaction in Sweden. These studies shed light on the importance of critical timing of snowmelt, soil moisture deficits and precipitation (Brandt et al., 1987; Lindström, 1990). An example from the Trängslet drainage basin in River Dalälven is shown in figure 5.

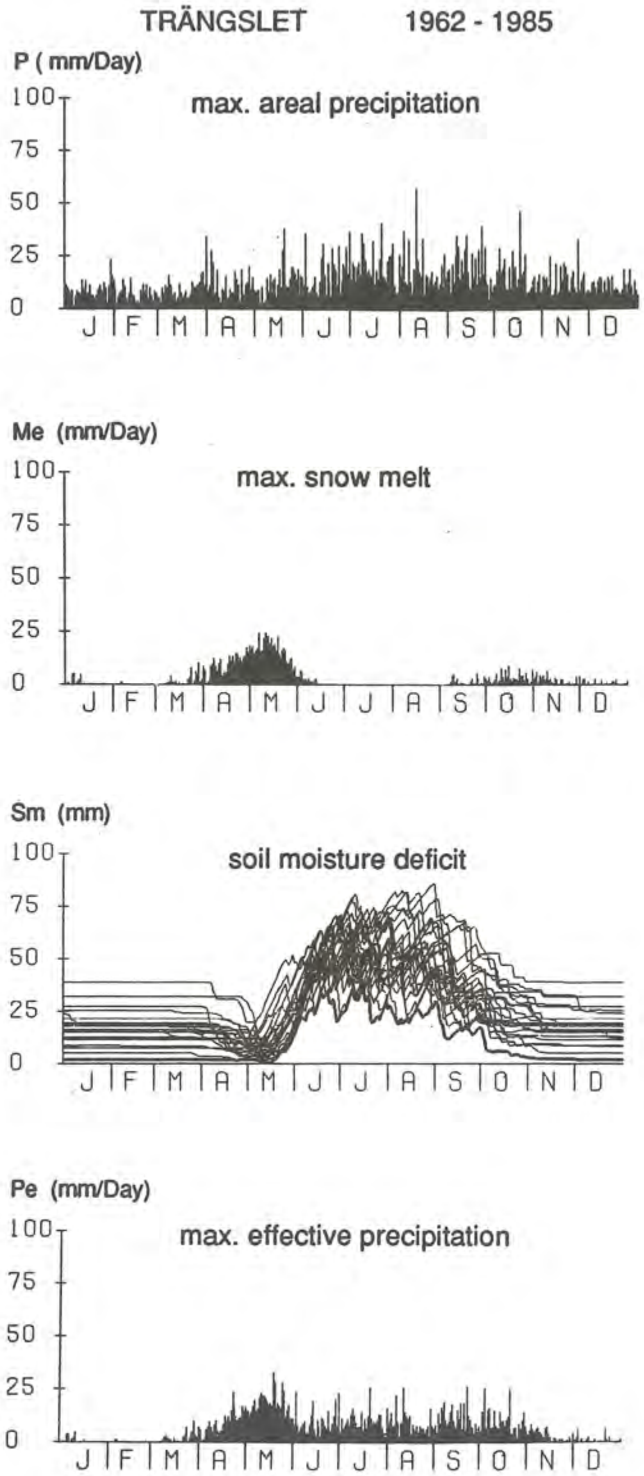

Figure 5. *Maximum daily areal precipitation and model simulations of maximum snow melt, soil moisture deficit and maximum runoff generating precipitation (effective precipitation) for the Trängslet basin (4 483 km^2) in River Dalälven over the period 1962 - 1985. The figure illustrates the importance of timing of the hydrological processes for flood generation. Note that all scales are identical.*

The committee first investigated the possibility to use a rather simplified model approach. This worked quite well for smaller systems in Northern Sweden (Bergström et al., 1989). It was, however, found that this method would be inappropriate when applied to Southern Sweden, where soil moisture deficits are substantial in summer. It was also found inadequate for the modelling of snowmelt floods and for applications to the large river basins of the North where spring or even summer conditions may prevail in the lowlands while there still is winter in the mountains. This means that one event may result in snow accumulation in the upper parts of the basin and flood generation in the mid parts, gradually reduced by the increasing soil moisture deficits in the lower parts.

According to the guidelines the simulation of design floods shall consequently be based on a relatively complete hydrological model. This means that the model must include routines for snow accumulation and melt, soil moisture accounting and response of the basin to effective precipitation. Routines are also required for the distribution of the processes according to the elevation above sea level. Finally the model must have documented performance in order to yield credible results. The established model in Sweden, that meet these requirements, is the HBV model (Bergström, 1976), but other models of equal (or better) performance are also accepted. Input data to the HBV model are daily totals of precipitation, mean daily air temperatures and estimates of potential evapotranspiration, normally monthly averages. Output are daily values of discharge or inflow to a reservoir.

The committee decided that the design simulation shall start with a snowpack with the estimated return period of 30 years. This means that extrapolation to return periods far beyond the period of record is avoided. The frequency analysis is based on snowpacks that are generated by the hydrological model. The areal distribution of the design snowpack within the basin is the same as the one found during the most extreme of the years included in the model simulation, and the starting date is the last date of culmination of any snowpack in the same record. The technique means that the hydrological model has to be calibrated before the snowpack can be extracted for the statistical analysis. The calibration period is approximately ten to fifteen years.

Reservoir operation

The Swedish hydroelectric system is complex with a large number of interacting reservoirs. The operation of these has to be considered when modelling the response of the system. Generally the reservoirs are emptied before the onset of snowmelt and refilled during summer and autumn to a degree that depends on the climatological conditions during that specific year. When analysing reservoir response to design floods one may consequently assume that the reservoirs are emptied to a degree that is realistic when a large snowpack is experienced. A reservoir operation strategy, that is specific for each dam, is worked out in cooperation with the river regulation enterprise of the specific river. This strategy is followed when routing the inflow hydrograph through the reservoir during the flood event. The reservoirs are assumed full before the floods in late summer or in autumn (after August 1).

Experience has shown that a rain storm of the order of magnitude of the design precipitation sequence causes a more or less chaotic situation, which affects the transmission of electricity. It is therefore not realistic to assume that the water can be evacuated through the turbines of the power station. These are assumed to be shut down from the day of culmination of the precipitation sequence (day No. 9) and over the remaining days of the event.

Simulation technique

The computation of design floods starts in spring with the specified design snowpack, with reservoirs and lake water levels set at mean lowest levels for this time of the year and without soil moisture deficit. The most critical timing of the design precipitation is then found by a trial and error technique. This means that the corrected 14-days sequence is successively inserted, replacing the observed data, at all possible dates of a climatological record of at least ten years. The corresponding floods and water stage developments are simulated by the hydrological model and the reservoir operation strategies (Figure 6).

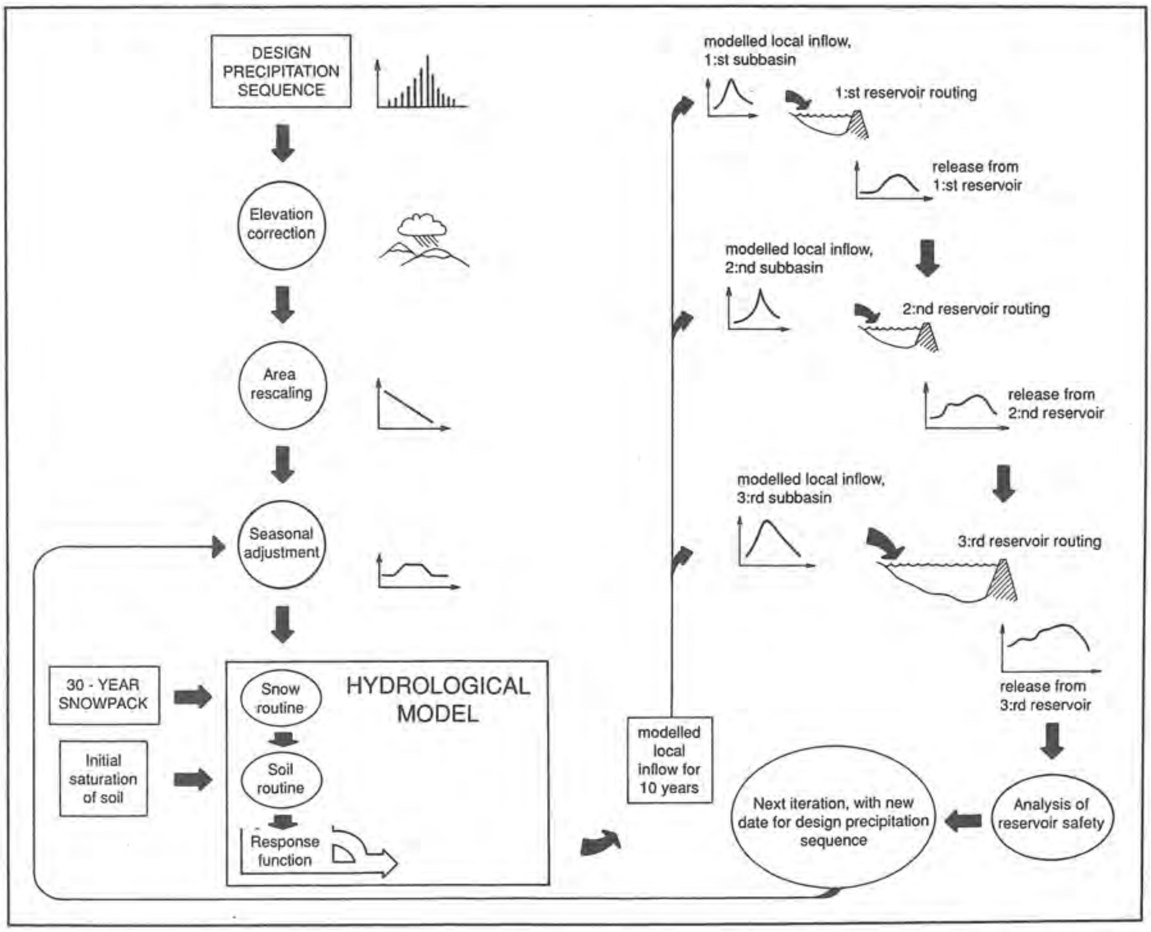

Figure 6. Schematic presentation of the flood simulation and routing through a system of reservoirs according to the Swedish guidelines for spillway design.

This methodology means that design snowmelt floods in spring and rainfloods other times of the year, or combinations of the two, are computed by one procedure.

One detail in the guidelines is that the air temperature in spring is lowered by 3 °C on day No. 9 and on the remaining days of the design precipitation sequence in order to avoid unrealistic combinations of snowmelt and rainstorms. Another detail is that precipitation adjacent to the inserted design sequence may be adjusted so that no floating 14 days totals will exceed the value of the design sequence.

The scaling of the design precipitation, according to basin area, may make local precipitation over a relatively small area more critical than precipitation over the entire river system. The committee therefore recommends that both the total inflow and the local inflow generated below an upstream dam or large natural lake shall be analysed.

The systematic search for the most severe flood situation within 10 years of climate observations means that, theoretically, 3650 simulations have to be carried out. Thanks to the relative simplicity of the hydrological model this is not an unsurmountable effort for modern desk top computers. Some of the simulations can also easily be identified as unnecessary and can therefore be omitted. The dam safety criterion is that all dams of the system must be able to withstand the most severe of the simulated inflows. Figure 7 shows how the simulated inflow peaks are distributed in time for a single reservoir, Torrön, in River Indalsälven. An example of the routing of one of the simulated inflow hydrographs through the same reservoir is shown in Figure 8.

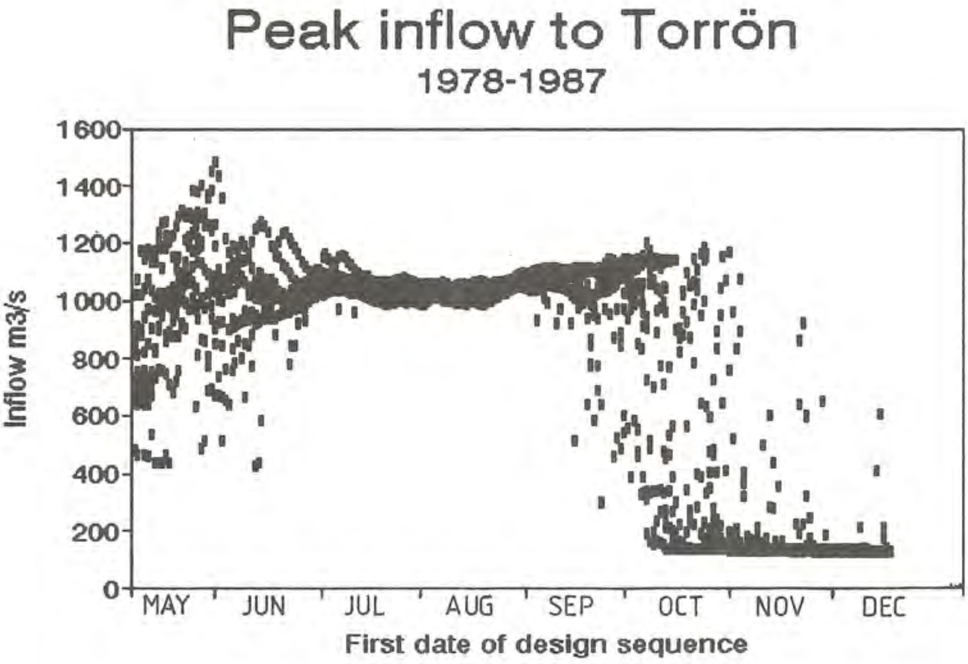

Figure 7. Distribution in time of the peak inflow to the Torrön reservoir according to the Torrön reservoir according to the iterative design simulation procedure.

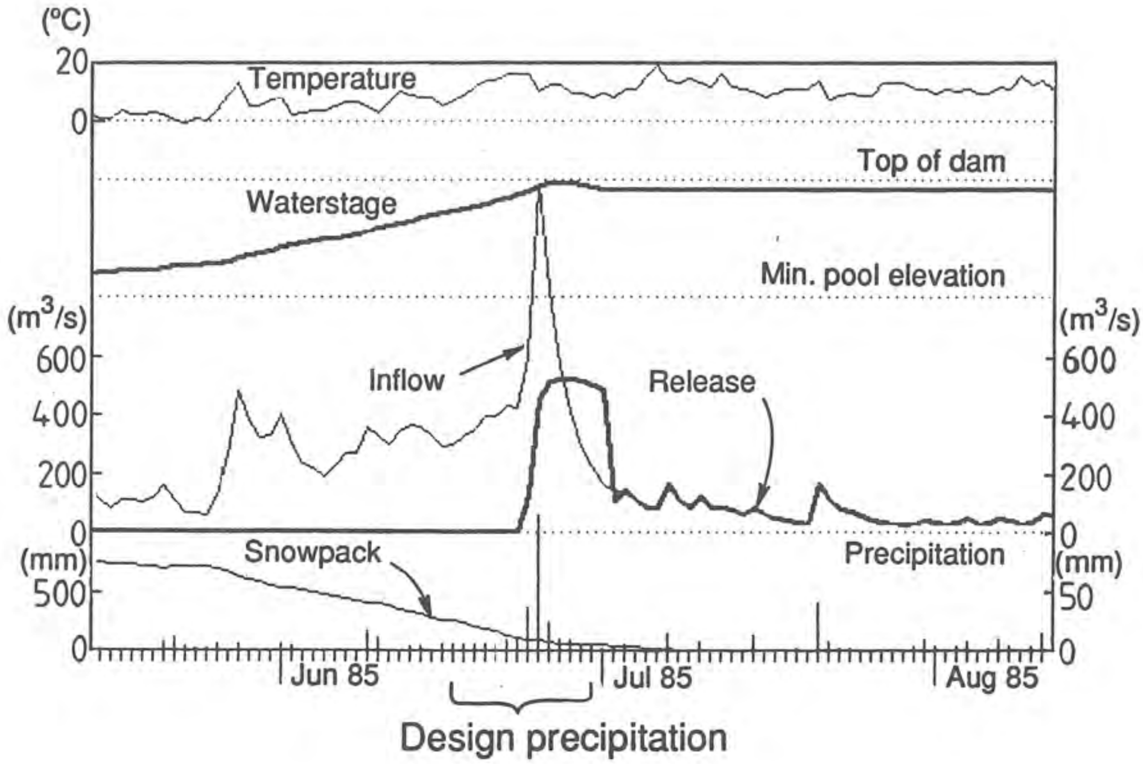

Figure 8. Example of the most critical inflow simulation and reservoir response in the single reservoir system of Torrön, Upper Indalsälven in Northern Sweden.

PROBABILITY AND CONSISTENCY

It is an urgent but difficult task to assess, or at least try to get an idea of, the probabilities of floods computed according to this type of guidelines. A lot of effort was spent on verification and control. First of all, computed design floods were compared to actual observations in the rivers, and secondly, comparisons were made with estimated 10 000 years floods according to standard procedures for flood frequency analysis. Results can be found in the work by Bergström (1988) and Bergström et al (1989) but also in an enclosure to the final report by the committee. Historical observations were used in a study by Harlin (1989).

The highest observed floods in Swedish records, as an average, stay below 45 % of the design flood for spring conditions whereas the corresponding number for summer and autumn is 40 %. Single extreme values did not exceed 65 % of the design flood after investigation of some 2 800 station-years. The results from the frequency analyses vary greatly between distribution functions and have to be interpreted with great care. Nevertheless, the conclusion of the committee is that the design floods according to the suggested procedures have return periods that exceed 10 000 years, but probabilities can not be assessed closer than this. The extensive control computations show no signs of regional inconsistencies.

DISCUSSION

The combination of a hydrological model and reservoir operation strategies has many advantages. The most important is that the significant flood generating factors can be combined in a realistic way and that extreme conditions with low probabilities can be reached without too much extrapolation of any one of them. Emphasis is put on critical timing. The variability of the seasonal patterns over a large basin is accounted for both as concerns the hydrological processes and the reservoir operation. The most important disadvantage is that a close specification of probabilities is difficult. One can, however, question whether this is possible with any of the existing methods for spillway design floods in use today (see, for example, U.S. Department of Commerce, 1986).

The use of reservoir operation strategies opens the possibility of mastering the design floods in a flexible way for the whole system. This means that problems in a river system can be solved by a combined strategy based on flood damping by temporary storage in some reservoirs and increased spillway capacities in others. It also means that the design flood analysis must be integrated for the entire river system and carried out in close cooperation with the river regulation enterprises.

The guidelines strictly prescribe most of the climatological and hydrological conditions for the computation of the design flood. This is, of course, a compromise, as many local effects can not be considered. In the two companion papers the sensitivity of the results to these assumptions is analysed further. The handling of the hydrological model introduces a main source of uncertainty. First of all, the results depend on the choice of model, and this is why a model with documented performance is required. Secondly, the results depend on how a specific model is calibrated. The sensitivity of the design floods to this uncertainty is analysed in the companion papers.

The conclusion by the Swedish Committee on Spillway Design, after five years of work, is that the suggested method meets the requirements. There is also a general consensus among the most important dam owners that these guidelines are realistic and will result in an acceptable and reasonable level of safety for the dams of the Swedish hydroelectric power system. The industry has now initiated a comprehensive control program where the safety of all major dams, with respect to design floods, is analysed.

ACKNOWLEDGEMENTS

The work of the Swedish Committee on Spillway Design and related climatological and hydrological investigations has been funded by the Swedish Association of River Regulation Enterprises (VASO). Thanks are also due to our many colleagues in Sweden and abroad who have contributed to the vivid debate on this important hydrological issue.

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II

Spillway design floods in Sweden, Part 2: Sensitivity analysis of inflow to single reservoirs

by

J. Harlin and G. Lindström

Hydrological Sciences Journal
(Submitted)

SPILLWAY DESIGN FLOODS IN SWEDEN

Part 2. Sensitivity analysis of inflow to single reservoirs

by

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ABSTRACT

The recently developed guidelines for design flood calculation in Sweden are based on simulation with a hydrological model. Several meteorological and hydrological conditions are prescribed as input to the model. The sensitivity of the floods according to the new guidelines with respect to these prescriptions is checked as well as the sensitivity to the modelling procedure. In this paper, the HBV hydrological model is used. It is shown that the most sensitive input factor is the design 14-day precipitation sequence. For autumn floods the scaling relation between the precipitation sequence and the generated floods is in the order of 1:1. For spring floods on the other hand, snowmelt influences the flood magnitude and reduces the sensitivity to precipitation to about 1:0.6. Soil moisture modelling was important in basins with high evapotranspiration. Model calibration could have large effect on the design flood magnitude. The most sensitive parameters were the high flow recession coefficient K_0 and the routing parameter MAXBAS.

DÉBITS DE CRUE DE PROJET POUR LES DÉVERSIONS EN SUÈDE

Deuxième partie: analyse de la sensibilité de l'afflux vers des réservoir simples

RÉSUMÉ

Les directives récemment développées pour le calcul des débits de crue de projet en Suède son basées sur une simulation par un modèle hydrologique. De nombreuses conditions météorologiques et hydrauliques sont prescrites comme données du modèle. On vérifie la sensibilité des debits ainsi que la sensibilité de la procédure de modèlisation selon les nouvelles directives, en respectant ces prescriptions. Dans cette étude, le modèle hydrologique HBV est utilisé. Il a été démontré que le facteur d'entrée le plus sensible est la série de précipitation de projet de 2 semaines. Pour les flots d'automne, la relation d'échelle entre la série de précipitations et les débits obtenus est de l'ordre de 1 pour 1. Pour les flots de printemps par contre, la fonte des neiges influence l'amplitude des débits et réduit la sensibilité des précipitations à environ 1 pour 0,6. La

modèlisation de l'humidité du sol fut importante dans les bassins à forte évapotranspiration. Le calibrage du modèle pourrait avoir un effet important sur l'amplitude du débit de crue de projet. Les paramètres les plus sensibles furent le coefficient K_0 de reflux des hautes eaux, et le paramètre d'acheminement MAXBAS.

INTRODUCTION

New guidelines for spillway design in Sweden were recently suggested by the Swedish Committee on Spillway Design (Flödeskommittén, 1990). The meteorological and hydrological conditions are given in the guidelines. An extreme areal precipitation sequence over 14 days is the foundation of the guidelines, and the design floods shall be simulated by using a hydrological model. The design precipitation sequence is specific for each one of five climatological regions in Sweden, and it is adjusted for basin area, altitude and time of the year. The simulations start from an initial snow pack with a return period of 30 years. The precipitation sequence is inserted, replacing the observed values, at all possible dates in a climate record of at least ten years.

A full description of the guidelines is given in the companion paper by Bergström et al. (1992). This paper presents a sensitivity analysis of the guidelines for four head water basins, in different hydrological regimes of Sweden. The sensitivity analysis was performed by evaluating the relative change of the generated flood peaks to relative changes in the input conditions, model parameters etc. Singh (1977) used this method for assessing the sensitivity of some runoff models to errors in rainfall excess. It is also a commonly used method in sensitivity analyses of model parameters (see, for example, Rogers et al., 1985, and Calver, 1988). In this paper no attempt is made to estimate the uncertainty range of the design floods. If an uncertainty range is sought for, a Monte Carlo approach similar to that used by Hornberger et al. (1986) could be more appropriate. However, the objective with this study was to determine which assumptions and factors in the guidelines that have the greatest influence on the generated flood peaks. In a companion paper by Lindström and Harlin (1992), these factors were studied further in a sensitivity analysis of the water stage development in a reservoir system.

METHODOLOGY

The HBV hydrological model

The HBV model (Bergström, 1976) is a well established runoff model in Sweden. It has been used for more than a decade for operational forecasting in a large number of basins. Since the release of the new guidelines for spillway design it is also being used for design flood simulation. The model has routines for snowmelt, soil moisture accounting and evapotranspiration, control of runoff response and a transformation function. It is usually run with daily means of air temperature and precipitation, and with monthly standard estimates of potential evapotranspiration. If the air temperature T exceeds the threshold temperature TT, snowmelt is calculated by the degree-day method:

$$\text{Melt} = \text{CFMAX} \cdot (\text{T} - \text{TT}) \tag{1}$$

where Melt = snowmelt, mm day^{-1}
CFMAX = degree-day factor, mm $°C^{-1}$ day^{-1}

The snowpack is assumed to retain liquid meltwater until the unfrozen water content reaches 10 % of the total snowpack water equivalent. If the air temperature is below the threshold temperature, unfrozen water in the snowpack refreezes according to:

$$\text{Refreeze} = \text{CFR} \cdot \text{CFMAX} \cdot (\text{TT} - \text{T}) \tag{2}$$

Refreeze = refreezing meltwater, mm day^{-1}
CFR = refreezing factor.

Water from precipitation or snowmelt enters the soil routine (Figure 1). The precipitation (dP) is portioned into a contribution to runoff (dQ) or an increase in soil moisture (S_{sm}). The soil routine gives a small contribution to runoff from rain or snowmelt when the soil is dry and a large contribution for wet conditions. The actual evapotranspiration (E_a) is a function of the soil moisture conditions. The evapotranspiration increases with increasing soil moisture storage, until it reaches its potential value E_p.

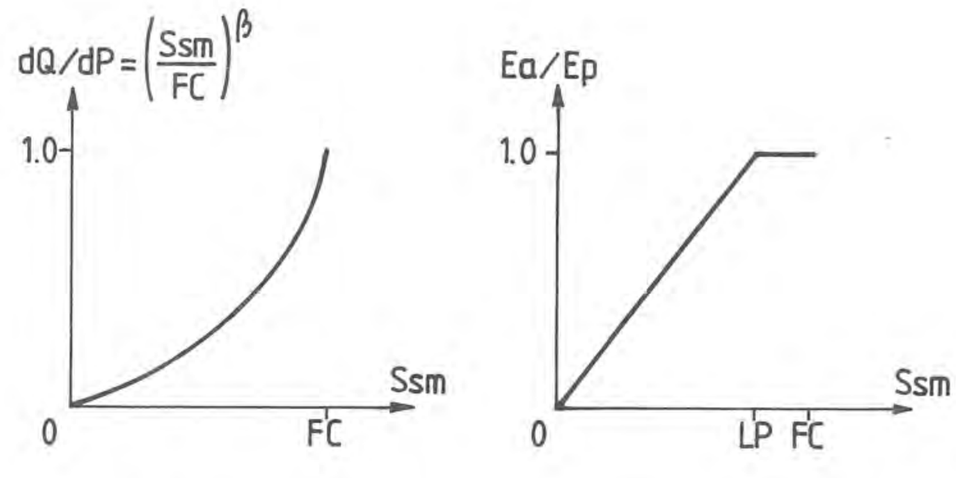


Figure 1. The soil moisture and evapotranspiration routines in the HBV model. FC is the maximum soil moisture storage, and LP is the soil moisture limit for potential evapotranspiration.

Runoff is modelled by two linked response tanks (Figure 2). The yield of water from the soil routine, the effective precipitation, is added to the storage (S_{uz}) in the upper tank. This tank is drained by two recession coefficients (K_0 and K_1), separated by a storage threshold (UZL). Water percolates from the upper tank and adds to the storage in the lower tank (S_{lz}) by the rate PERC. The lower tank also includes evaporation from and precipitation on lakes and it is drained by the recession coefficient K_2. The runoff is computed independently for each subbasin by adding the contributions from the upper and lower tanks. To account for the flood damping in the river, a simple routing transformation is made. This filter has a triangular distribution of weights with the base length MAXBAS days.

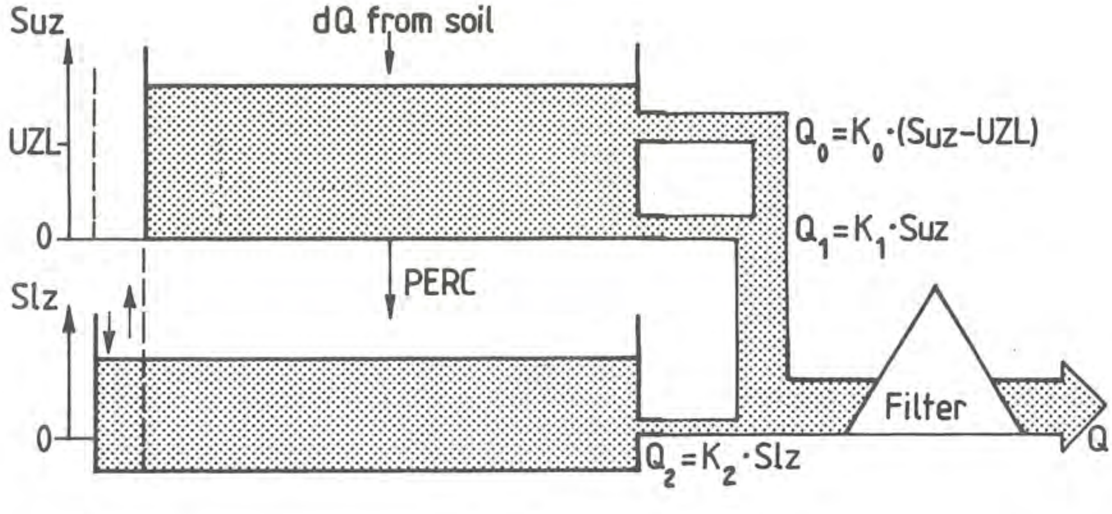


Figure 2. Schematic presentation of the runoff-response function of the HBV model.

Studied basins

Four basins in different parts of Sweden were selected: Tjaktjajaure, Torrön, Trängslet and Blankaström (Figure 3), representing different hydrological regimes. A hydropower reservoir is located at the point of interest in each of the basins. Torrön and Tjaktjajaure are mountainous, located partly above the timber line. Runoff is dominated by snowmelt floods in spring and rainfall floods in autumn. In Torrön, floods are also sometimes experienced during winter. Trängslet belongs to an inland regime and most of the basin is forested. The runoff follows a clear seasonal pattern. Snowmelt floods dominate, but large rain floods are occasionally experienced in summer and autumn. Blankaström belongs to the milder climate of southern Sweden. It is partly cultivated and the terrain is flat to rolling. The evapotranspiration is higher than in the other three basins. The snow pack in the short winters is fairly small and the seasonal runoff pattern is less regular. The HBV model was set-up and calibrated against the observed inflow to each of the reservoirs. Special attention was given to the parameters that most directly affect the peak flow.

BASIN	1 TJAKTJAJAURE	2 TORRÖN	3 TRÄNGSLET	4 BLANKASTRÖM
RIVER	Lilla Lule älv	Indalsälven	Österdalälven	Emån
AREA	2267 km^2	1370 km^2	4483 km^2	3446 km^2

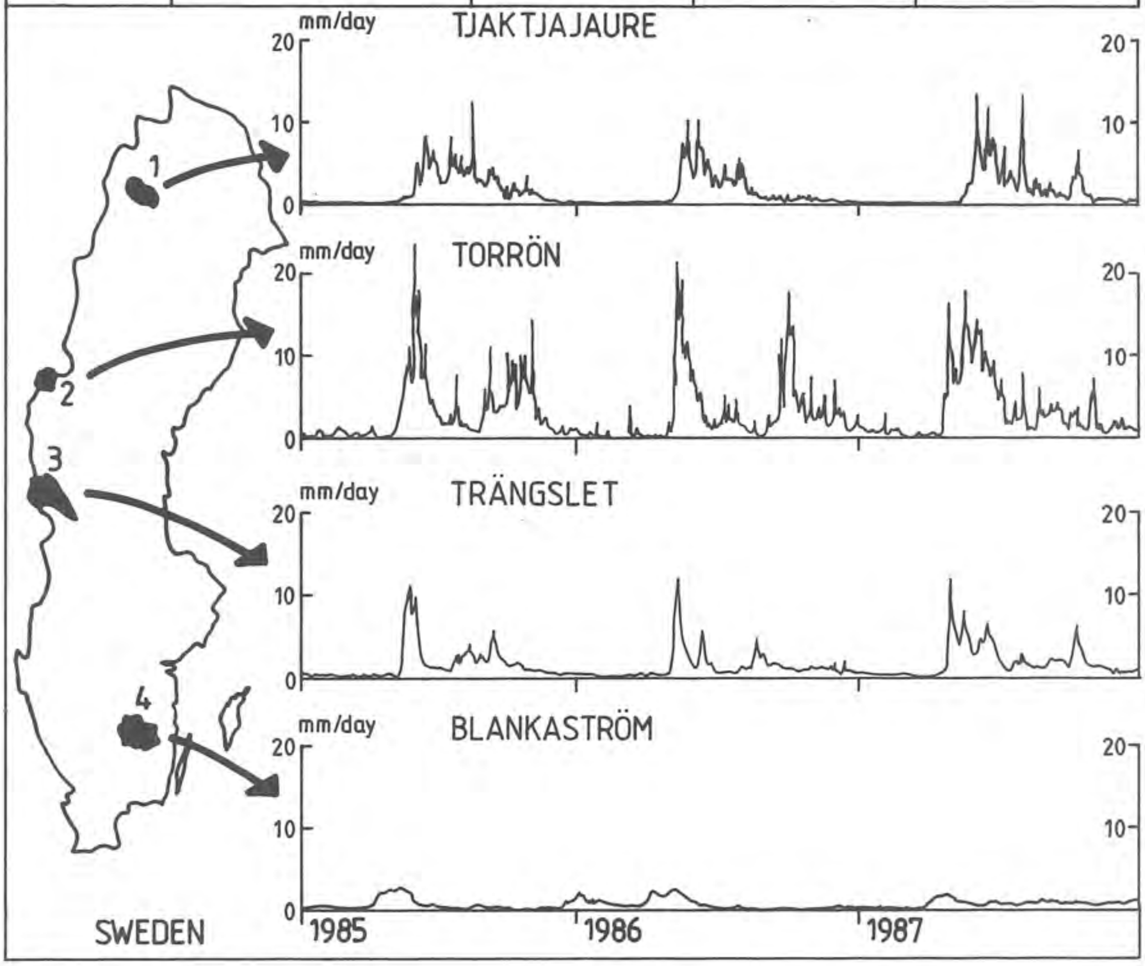


Figure 3. Location, key data and examples of specific runoff for the studied basins.

Sensitivity to meteorological and hydrological prescriptions

The new guidelines prescribe that all reservoirs shall be filled from 1 August. In this paper the period before 1 August is called spring, and the period after that date is called autumn. The largest inflow peaks in spring and autumn were first calculated in a reference run, for each reservoir, by strictly following the guidelines. The sensitivity of the largest inflow peaks was then studied by changing one factor at a time in the design calculation, while keeping all other factors as in the reference run. No reservoir operation strategies were formulated, as only inflow peaks were studied. The details of the design guidelines can be found in the companion paper by Bergström et al. (1992).

The sensitivity to the design precipitation amount was checked by rescaling the whole sequence. This is also equivalent to changing the areal adjustment factor. The altitude adjustment factors were altered by ± 10 % in relation to their reference values (Table 1). The seasonal adjustment was altered according to Figure 4. The shape of the 14-day precipitation sequence was varied, without changing the total volume and with the 24-hour maximum intact (Figure 5). Runs were made without any reduction of the observed precipitation, before or after the inserted design sequence, in the events when some floating 14 days total exceeded the design value.

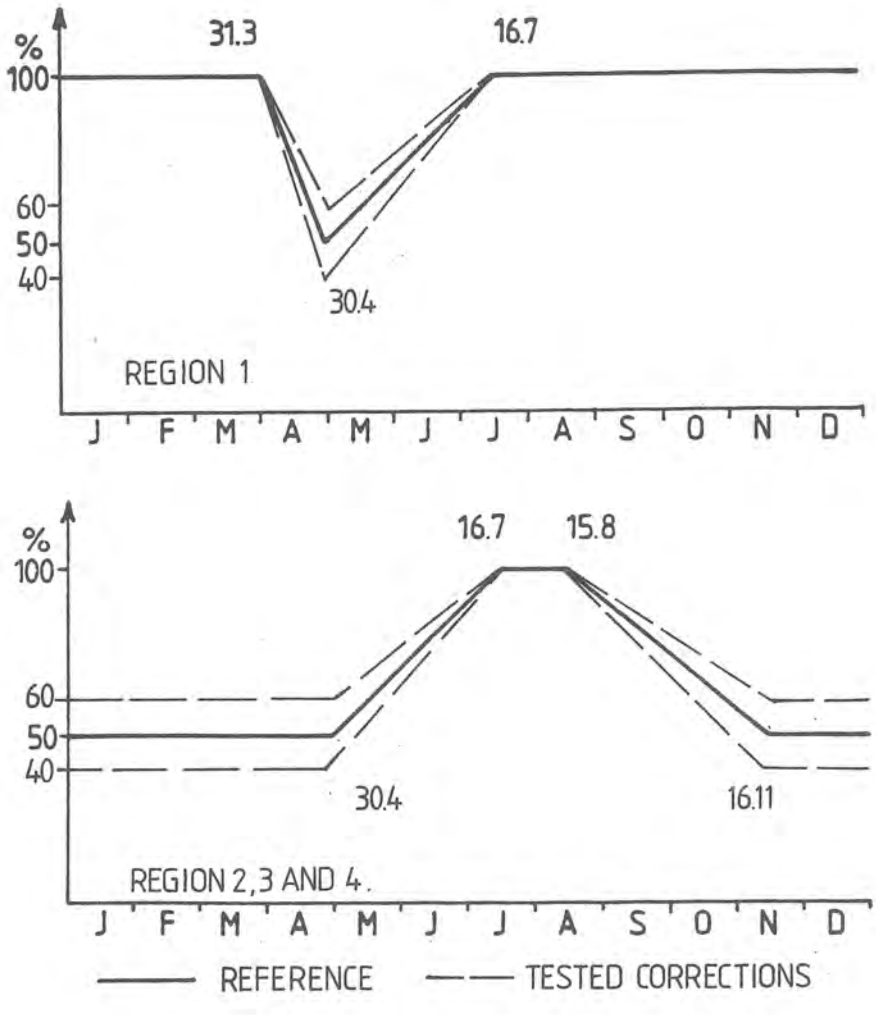


Figure 4. The alternative seasonal adjustment curves for the design precipitation sequence in the sensitivity analysis.

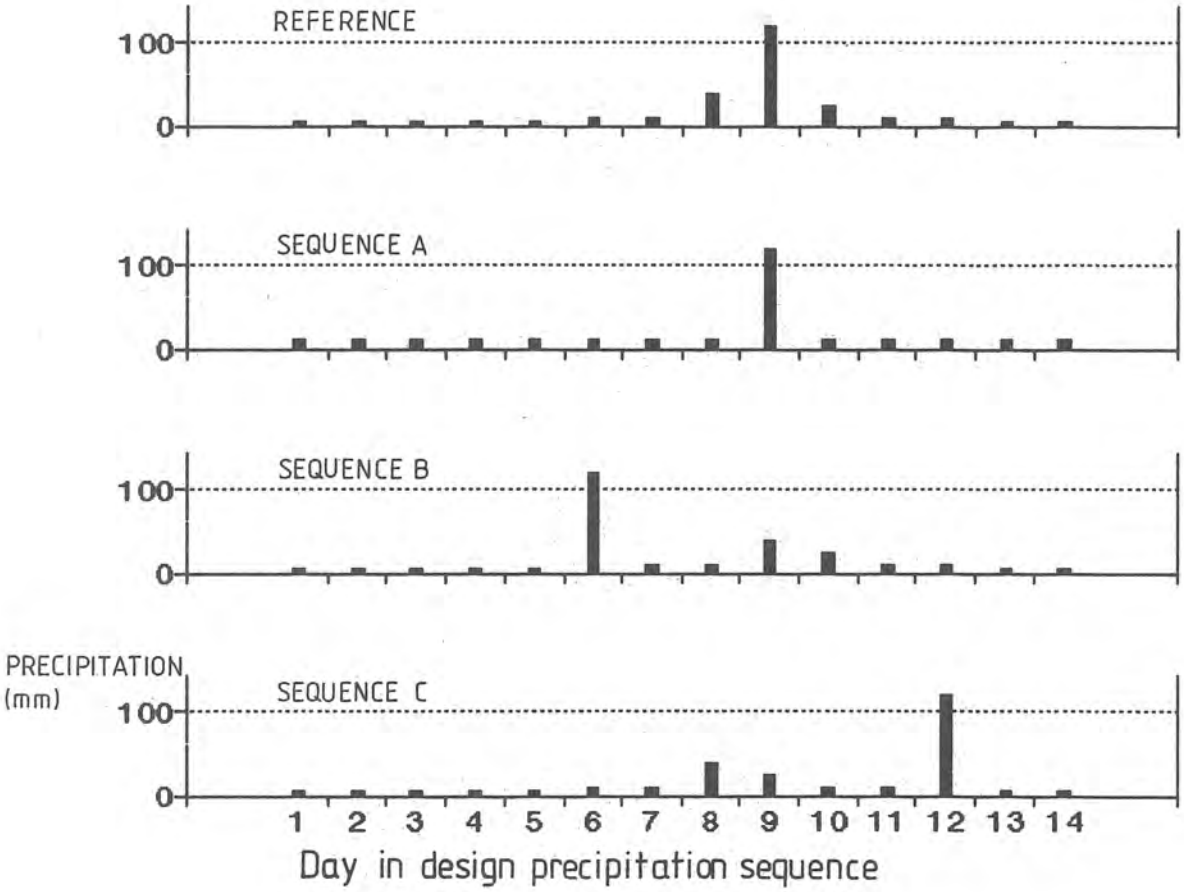


Figure 5. *The reference design precipitation sequence and the alternative arrangements in the sensitivity analysis.*

Table 1. *Altitude adjustments in the sensitivity analysis.*

Basin	Mean basin altitude (m)	Rescaling of design precipitation in % / 100 m above the reference altitude				
		Reference altitude (m)	Sens. test	Reference	Sens. test
Tjaktjajaure	971	500	+9	+10	+11
Torrön	644	500	+9	+10	+11
Trängslet	670	600	+4.5	+ 5	+ 5.5
Blankaström	[1]	[1]	0	0	0

[1] No adjustments for altitude in this region.

Simulations were made without the prescribed reduction of the temperature by 3 °C from day 9 to 14 in the design sequence. An initial soil moisture deficit of 10 % was tested, instead of the prescribed initial saturation. The sensitivity to the snow pack was analyzed in three ways. Firstly, the snow pack was varied between return periods of 2 and 100 years. Secondly, the snow distributions of the four years with largest snowpacks were tried, and thirdly, the starting date of the design calculation was changed +- 10 days. There is no general rule for choosing frequency distribution and parameter estimation method. To check the uncertainty in the frequency analysis the analysis was made with both the methods of moments and of maximum likelihood for the Gumbel, Lognormal 2 and 3 parameter distributions. The Swedish spillway design floods shall be simulated with a full hydrological model including soil moisture accounting and evapotranspiration. Simulations were also made with a simplification of the model, in which no evapotranspiration was assumed.

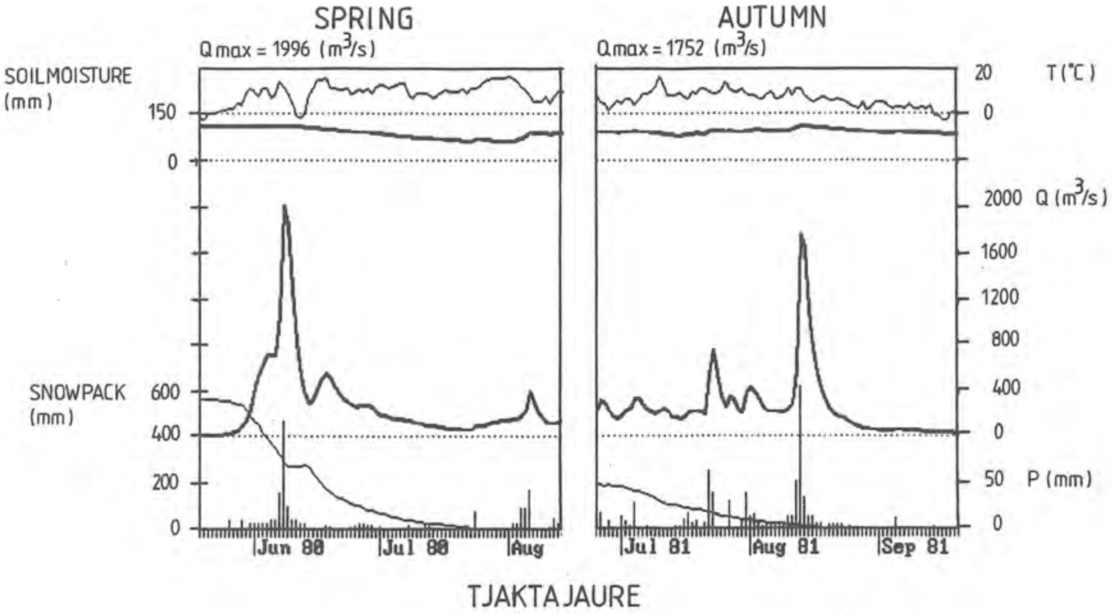
Sensitivity to modelling aspects

The suggested step-length for moving the precipitation sequence through the observed climate records was varied, as was the suggested number of years in the simulation. To evaluate the effect of uncertainties stemming from the calibration of the HBV model, five parameters were analyzed. Based on experience from manual and automatic calibration at the SMHI, the parameters CFMAX, FC, K_0, UZL and MAXBAS were chosen for the sensitivity analysis, as being the most important for peak flow simulation. MAXBAS was increased by 1 day in all basins. This is a very large change for a small basin such as Torrön, but a rather small change for the slowly reacting Blankaström basin.

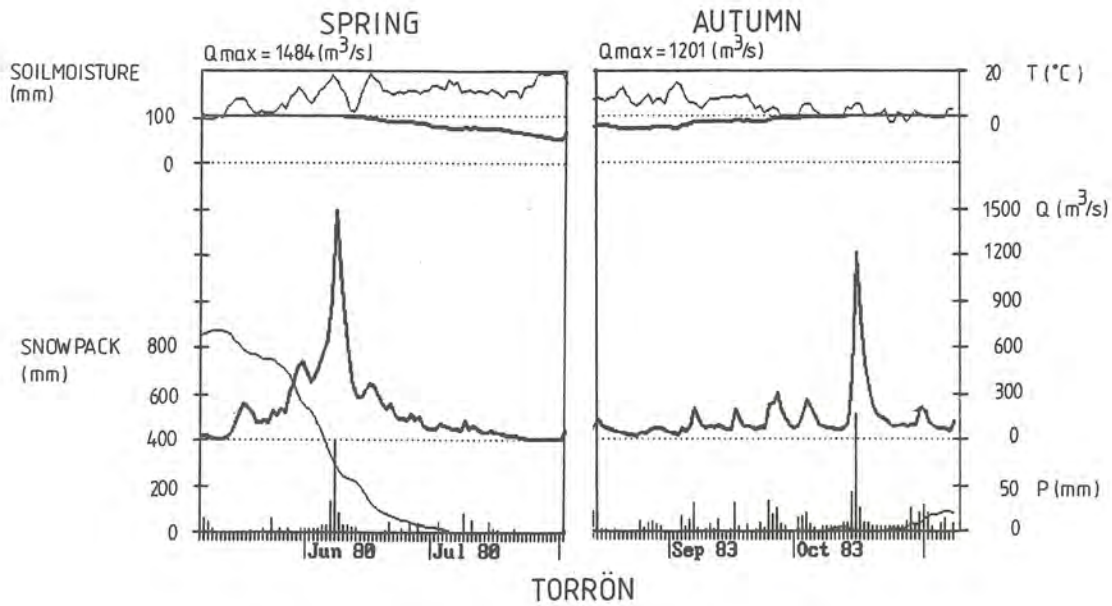
RESULTS AND DISCUSSION

Largest inflow peaks

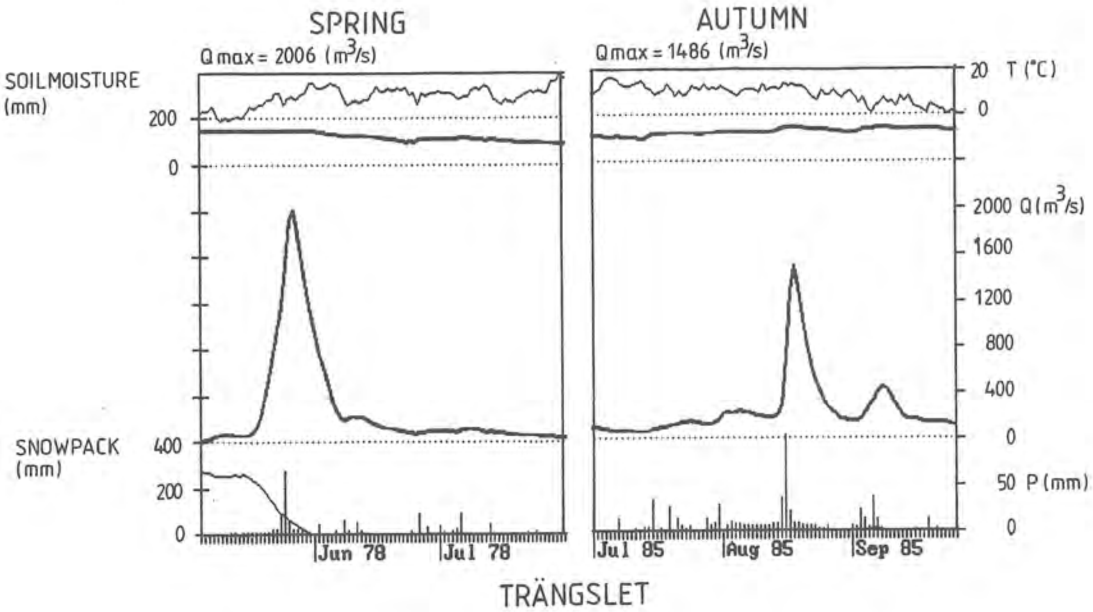
Figure 6 (a, b, c and d) shows the largest spring and autumn floods for the four basins. The largest floods occurred in spring, when the design precipitation sequence coincided with snowmelt. In Tjaktjajaure, Trängslet and Blankaström the largest autumn floods occurred in August, when the design precipitation was at its maximum value. However, in Torrön, a snowmelt situation combined with the rainfall sequence in October resulted in the highest autumn flood.


SPRING
Qmax = 1996 (m3/s)
AUTUMN
Qmax = 1752 (m3/s)
SOILMOISTURE (mm)
SNOWPACK (mm)
T (°C)
Q (m3/s)
P (mm)
Jun 80
Jul 80
Aug
Jul 81
Aug 81
Sep 81
TJAKTAJAURE

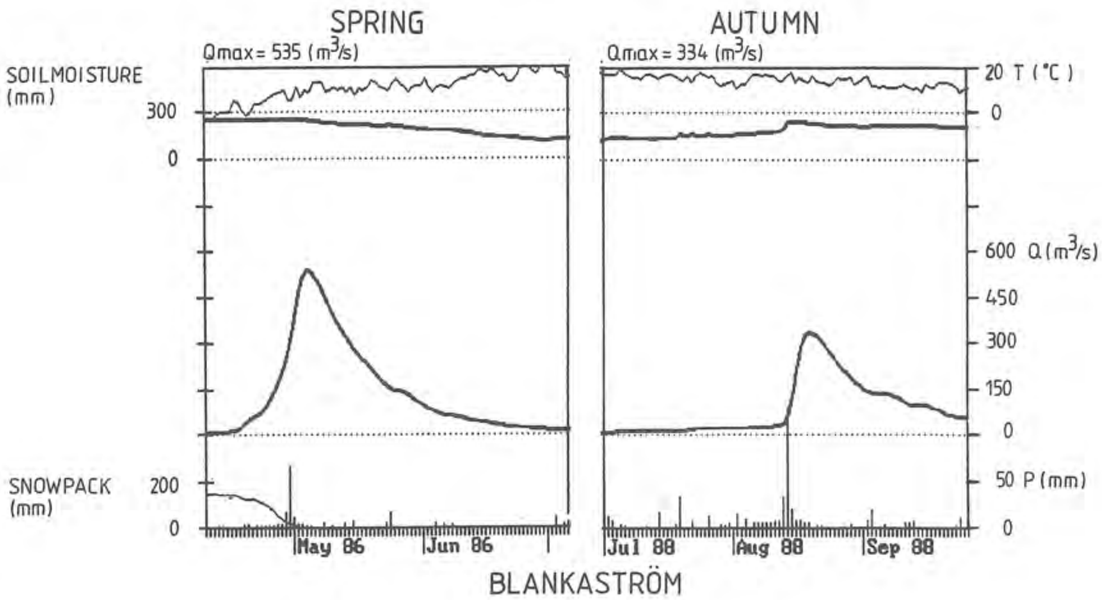
6 a.


SPRING
Qmax = 1484 (m3/s)
AUTUMN
Qmax = 1201 (m3/s)
SOILMOISTURE (mm)
SNOWPACK (mm)
T (°C)
Q (m3/s)
P (mm)
Jun 80
Jul 80
Sep 83
Oct 83
TORRÖN

6 b.



6 c.



6 d.

Figure 6. ***The largest spring and autumn floods when applying the Swedish design guidelines to the basins a) Tjaktjajaure, b) Torrön, c) Trängslet and d) Blankaström.***

Sensitivity to preset design precipitation

Changes in the design 14-day precipitation sequence had a large impact on the resulting floods, Figure 7. The influence of the 14-day rainfall amount was larger in autumn than in spring, since the spring floods are caused by combinations of snowmelt and rainfall. The effect of rescaling the rainfall was greatest in the basins with large amplitudes in soil moisture, primarily Trängslet and Blankaström. This is because the flood magnitude depends on the antecedent soil moisture conditions.

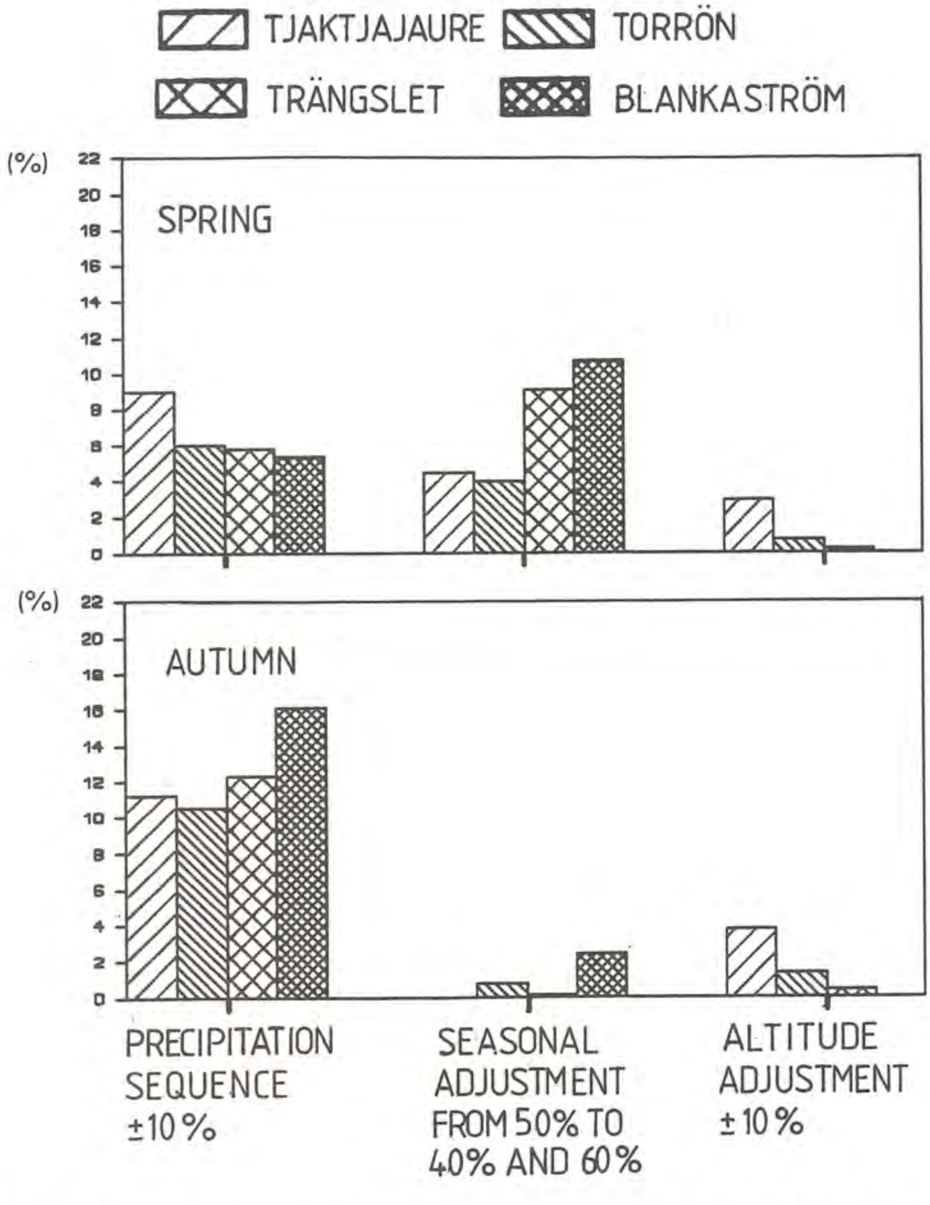


Figure 7. *The largest absolute sensitivity of highest flood peaks due to changes in the design precipitation.*

The seasonal adjustment curve (Figure 4) for the storm sequence was also important. The spring floods were more sensitive than the autumn floods, and the southern basins were the most sensitive ones (Figure 7). The explanation is that the largest spring floods in the south occur earlier, when the seasonal reduction is large. In autumn the design floods occurred before the seasonal reduction of the precipitation had any large effect, hence the low sensitivity. Figure 7 further shows that even a moderate change in the altitude rescaling, from 10 to 11 % per 100 m above the reference altitude, had a considerable effect in Tjaktjajaure. All alternative shapes of the design precipitation sequence reduced the flood peak (Figure 8).

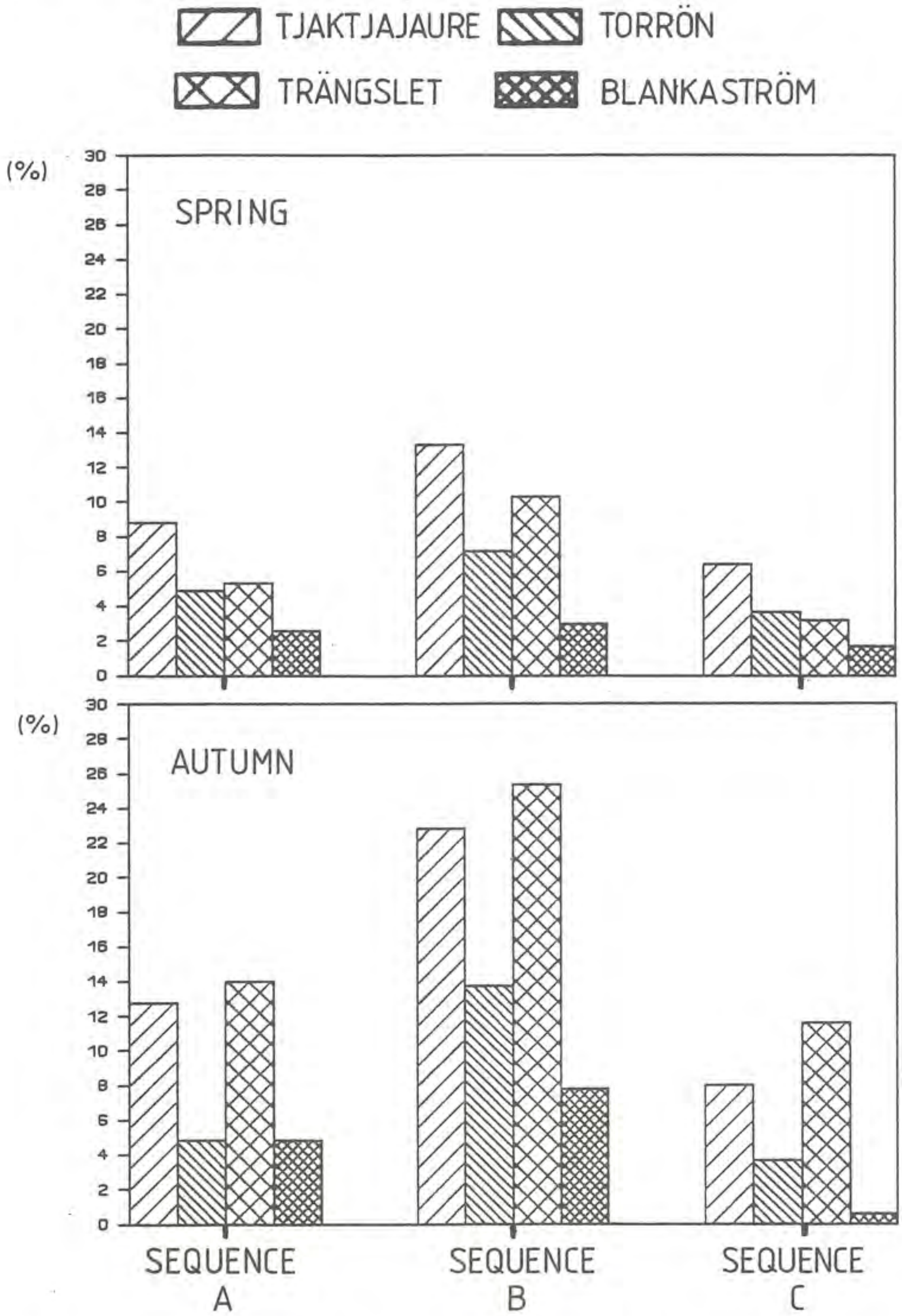


Figure 8. *The reduction in largest flood peaks due to changes in the shape of the design storm sequence.*

The results show that the precipitation that immediately precedes the day with the most intense rainfall (day 9), significantly influences the resulting flood peak. The adjustment of rainfall surrounding the design storm sequence had small influence on the results. The largest effect of omitting this adjustment was only 2 %.

Sensitivity to preset snowpack and soil moisture conditions

The effect of the return period of the design snow pack is shown in Figure 9. The 30-year snow pack gave 8 to 30 percent larger floods than did the snowpack of a normal year. When the return period was doubled to 60 years, the flood peaks increased with 10 % or less. The sensitivity to snow pack return period was largest in Blankaström. This basin has the least snow, and large variations from year to year (Figure 10). The water equivalent of the 30-year snowpack in any of the four test basins could change at the most with 14 % depending on the choice of frequency distribution and parameter estimation method.

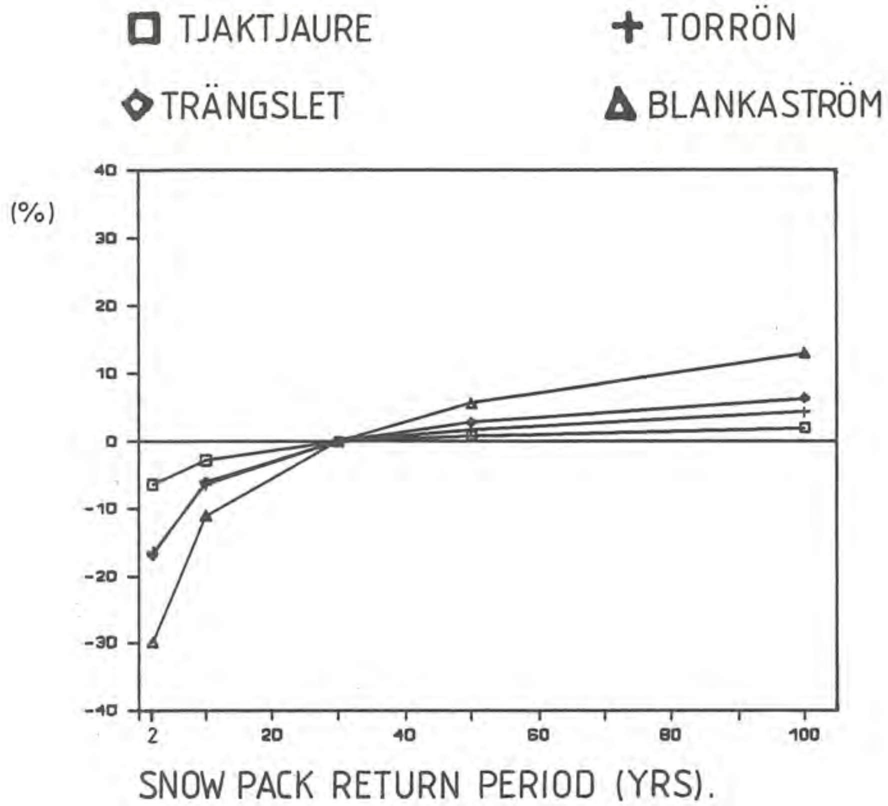


Figure 9. Change in largest spring floods peak versus the return period of the initial snow pack.

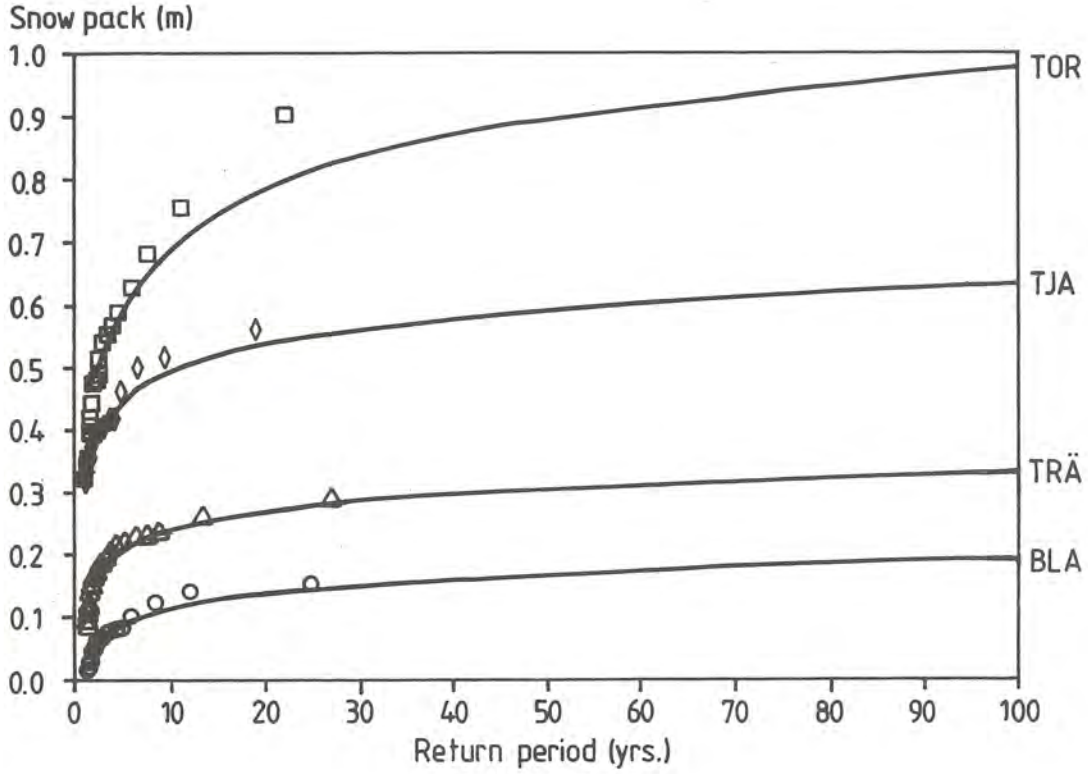


Figure 10. *Gumbel frequency distributions of annual maximum snow pack for the four studied basins. The observations are plotted with Weibuls plotting position formula. TOR = Torrön, TJA = Tjaktjajaure, TRÄ = Trängslet, BLA = Blankaström.*

Figure 11 gives the sensitivity of the spring floods to changes in the initial conditions; snow culmination date, snow pack size, snow distribution in the basin and initial soil moisture deficit. The autumn floods changed by less than one percent, when these initial conditions were varied by the same amount. Simplified model runs, without evapotranspiration, resulted in much larger floods in the southern basins, whereas almost no effect was seen in the north (Table 2). An increase of between one and six percent of the spring flood peak resulted when the temperature reduction from day nine to fourteen was omitted.

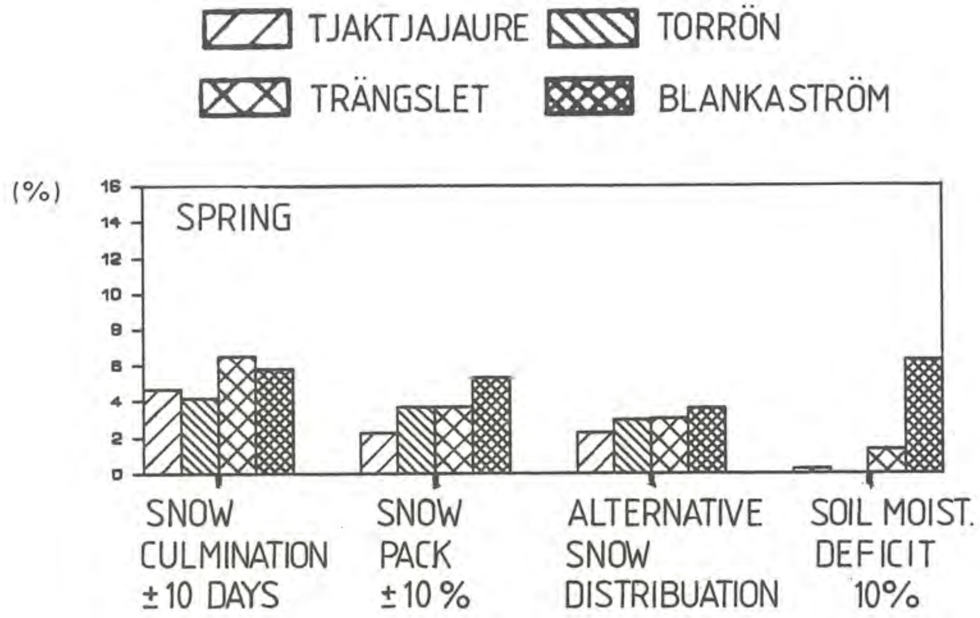


Figure 11. *The largest absolute sensitivity of highest flood peaks due to changes in the initial state.*

Table 2. *Largest floods generated without evapotranspiration in relation to the reference values (in percent).*

Basin	Spring	Autumn
Tjaktjajaure	+ 3.6	+ 8.2
Torrön	+ 0.5	+ 0.7
Trängslet	+ 0.5	+17.7
Blankaström	+12.2	+85.7

Sensitivity to modelling aspects

The timing of events was important. The most critical event was sometimes missed when the step length between rainfall sequence placings was increased. The largest underestimations due to this were about 2 %, 8 % and 12 % for step lengths of 2, 4 and 8 days, respectively. Figure 12 shows that the result also depended on the number of years in the simulation. The sensitivity to the studied model parameters is given in Figure 13.

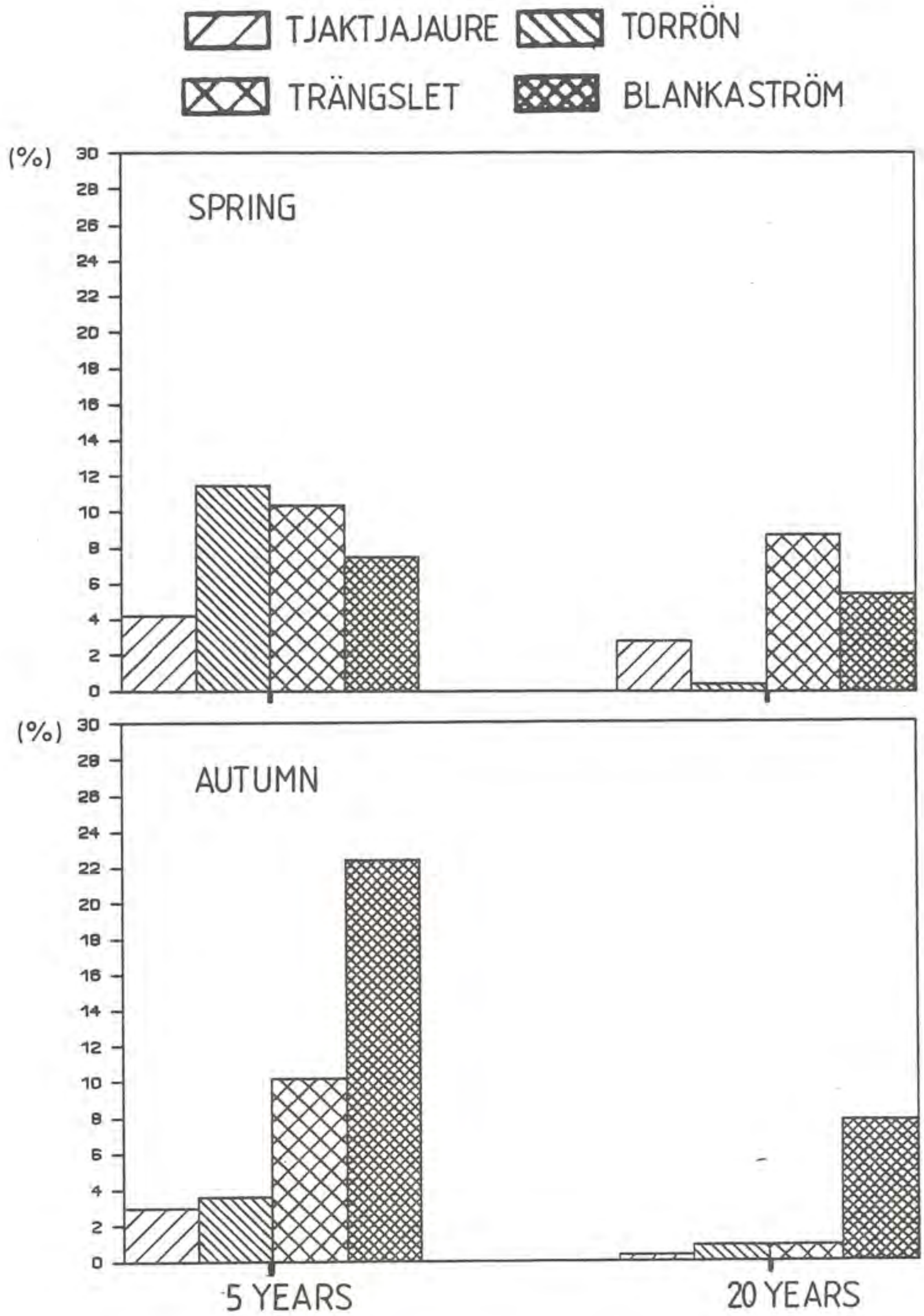


Figure 12. *The largest absolute sensitivity of flood peaks with respect to simulation period length. 5 YEARS refer to the largest difference between any two of four consecutive five-year periods, and 20 YEARS is the corresponding difference between two consecutive ten-year periods.*

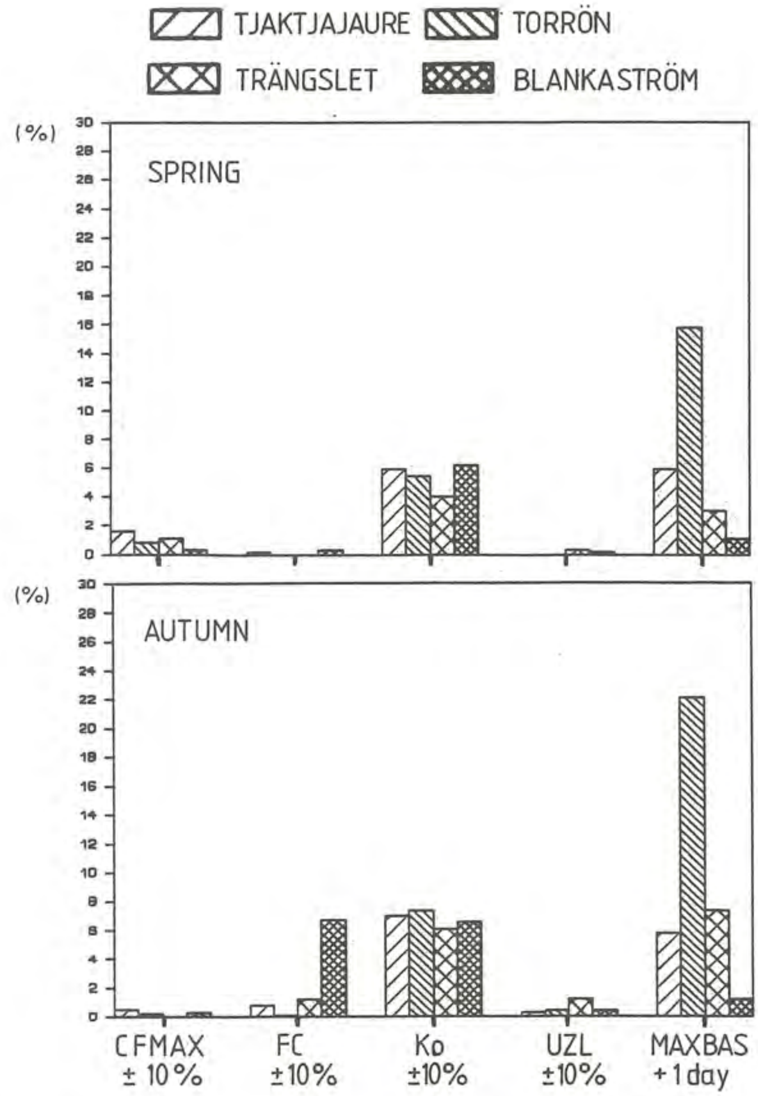


Figure 13. The largest change (in absolute values) in largest flood peaks due to changes in the HBV model parameters.

The highest recession coefficient K_0 was crucial for both spring and autumn floods. Changes in the soil moisture parameter FC had a large impact in the southern basin of Blankaström. The degree-day factor was less important, as changes in the parameter sometimes lead to a shift in the critical period. The small and quickly responding basin Torrön was very sensitive to an increase in the routing parameter MAXBAS from one to two days. A change in MAXBAS by one day is, however, a considerable change, and this parameter is normally one of the easiest to establish by model calibration. Not all parameters are equally well defined in a calibration. The performed analysis does not quantify the uncertainty in design floods stemming from uncertainties in the calibration of each parameter. It merely illustrates the sensitivity to changes in some important parameters, and indicates which ones that are most important to calibrate carefully.

CONCLUSIONS

The magnitude of the largest inflow peaks depends on a number of factors, of which the most important ones are preset by the guidelines. The design precipitation amount was found to be the most important of these preset factors. Alterations to the amount gave an effect of approximately 1:1 on the highest autumn floods and an effect of about 1:0.6 on the highest spring floods. None of the alternative precipitation sequences, gave as high floods as the prescribed sequence shape. On the other hand, the flood peaks were never reduced by more than 25 % when the sequence was modified. The seasonal adjustment curve was found to be important in the spring, but less important in the autumn. The adjustment of adjacent observed precipitation was not important.

The largest floods in the design calculation occurred when the precipitation sequence was combined with intense snowmelt in the spring. These spring floods may, however, not be the most critical for reservoir water stage as they may occur before the filling of the reservoir. The calculation must be complemented with a reservoir operation strategy for finding the most critical event. The number of years in the design simulation can affect the result, especially for spring floods. The guidelines recommend that at least 10 years shall be used. If possible, a longer period should be used, to reduce the influence of the choice of period. A daily step should be used for moving the precipitation sequence through the observed data, to ensure that the most critical event is found. However, when the critical location of the design storm was found, it was observed that this timing of events was often still the worst even after changing some factor in the calculation. This observation can be used to concentrate the trial and error procedure over the critical period, once the most critical timing of events has been found and only small changes are made.

The initial snow amount had a large influence on the spring peak flow, but not as large as that of the precipitation. The areal distribution of the initial snowpack was less critical, but the starting date in the calculation was important. None of the initial conditions affected the autumn floods. The peak flow parameter K_0 and the routing parameter MAXBAS were the most important model parameters. The small Torrön basin, with a quick response, was particularly sensitive to an increase in the routing parameter MAXBAS. The other parameters were less sensitive, except for the soil moisture parameter FC, which was important in the southern basin Blankaström.

ACKNOWLEDGEMENTS

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III

Spillway design floods in Sweden, Part 3:
Sensitivity analysis of water stage development
in a multi-reservoir system

by

G. Lindström, and J. Harlin

Hydrological Sciences Journal
(Submitted)

SPILLWAY DESIGN FLOODS IN SWEDEN

Part 3. Sensitivity analysis of water stage development in a multi-reservoir system

by

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ABSTRACT

A sensitivity analysis of the new Swedish guidelines for spillway design floods applied to the multi-reservoir system in River Ljusnan in Central Sweden is presented. The simulations were made using a model system based on the HBV model complemented with routing of inflow hydrographs through reservoirs according to given regulation strategies. The 14 560 km^2 river basin was divided into 16 subbasins. At three hydropower reservoirs the sensitivity of design water levels was studied with respect to changes in guideline prescriptions, reservoir operation strategies and model parameters. Alterations to the preset design precipitation, snow pack or the spillway capacity, all had significant impact on the highest water stage in all three reservoirs. Alterations to the reservoir operation strategies had less effect. The highest recession parameter in the HBV model, K_0, had a great influence on design water stages. After the filling of the reservoir there was a clear relation between inflow peak and maximum water stage. The study shows that it is difficult to assess the integrated effects of extreme precipitation, snowmelt, soil moisture status and reservoir operation in a system beforehand.

DÉBITS DE CRUE DE PROJET POUR LES DÉVERSOIRS EN SUÉDE

Troisième partie: analyse de la sensibilité de l'evolution du niveau d'eau dans un système à reservoirs multiples

RÉSUMÉ

Une analyse de la sensibilité des nouvelles directives suédoises pour les débits de crue de projet pour les déversoirs, appliquées au complexe à réservoirs multiples de la rivière Ljusnan en Suède centrale est présentée ici. les simulations furent exécutées en utilisant un système basé sur le modèle HBV complèté par les hydrographes d'acheminement du flot vers les réservoirs en tenant compte de stratégies de régulation données. Le bassin de la rivière d'aire 14 560 km^2 fut divisé en 16 sous-bassins. On a étudié la sensibilité des niveaux d'eau calculés sur trois réservoirs de centrales hydro-électriques, en respectant les changements prescrits par les directives, les stratégies d'exploitation du réservoir et les paramètres du modèle. Les modifications des précipitations de projet préselectionnées, la couche de neige ou la capacité du réservoir, tout a eu un impact significatif sur le niveau d'eau maximum dans les trois reservoirs. Les modifications sur les stratégies d'exploitation du réservoir ont eu moins d'effets. Le plus grand paramètre de reflux dans le modèle HBV, K_0, a eu une grande influence sur les hauteurs d'eau calculées. Après le remplissage du réservoir, il est apparu une relation nette entre le sommet de l'afflux et le niveau d'eau maximum. L'étude montre qu'il est difficile d'évaluer à l'avance dans le modèle les effets des précipitations extrêmes, de la fonte des neiges, des conditions d'humidité du sol et de l'exploitation du réservoir.

INTRODUCTION

New guidelines for computation of spillway design floods in Sweden were recently proposed by the Swedish Committee on Spillway Design (Flödeskommittén, 1990). The guidelines recommend the use of a hydrological model for combining critical but realistic hydrological and climatological conditions which have all been experienced, although not at the same time. The guidelines are based on an extreme 14 days design precipitation sequence and an initial snowpack with a return period of 30 years. The precipitation sequence is successively introduced at all possible dates within a period of at least ten years of climate observations. An important advantage of the guidelines is that they take can be applied to a system of reservoirs, by formulating an operation strategy for each reservoir in the system. All dams in a system must be able to withstand the most critical event simulated by this procedure.

Methods for design flood calculation and operational flood control differ between countries and sometimes even within one country (see, for example, ICOLD, 1988). Flood management models for multiple reservoir systems are often analysed and described in the literature (e.g. Yeh, 1985, and Unver et al., 1987). However, papers on simultaneous design flood simulations for multi-reservoir systems, as reported here, are hard to find.

A comprehensive description of the new Swedish guidelines is given in the companion paper by Bergström et al. (1992). In another companion paper Harlin and Lindström (1992) study the sensitivity in calculated design inflows to single reservoirs. Based on their results, some of the most critical guideline instructions and model parameters were chosen for further studies. This paper describes the application of the guidelines to existing reservoirs in the regulated River Ljusnan in Central Sweden. The sensitivity of the design calculations is then illustrated with respect to changes in the preset guidelines, the chosen regulation strategy, and the calibration of the hydrological model. The objective is to identify the most important assumptions and factors in the application of the new design guidelines to a system of reservoirs.

METHODOLOGY

The River Ljusnan basin

Design flood calculations were made for three reservoirs in the River Ljusnan system in Central Sweden: Lossen, Sveg and Arbråsjöarna (Figure 1).

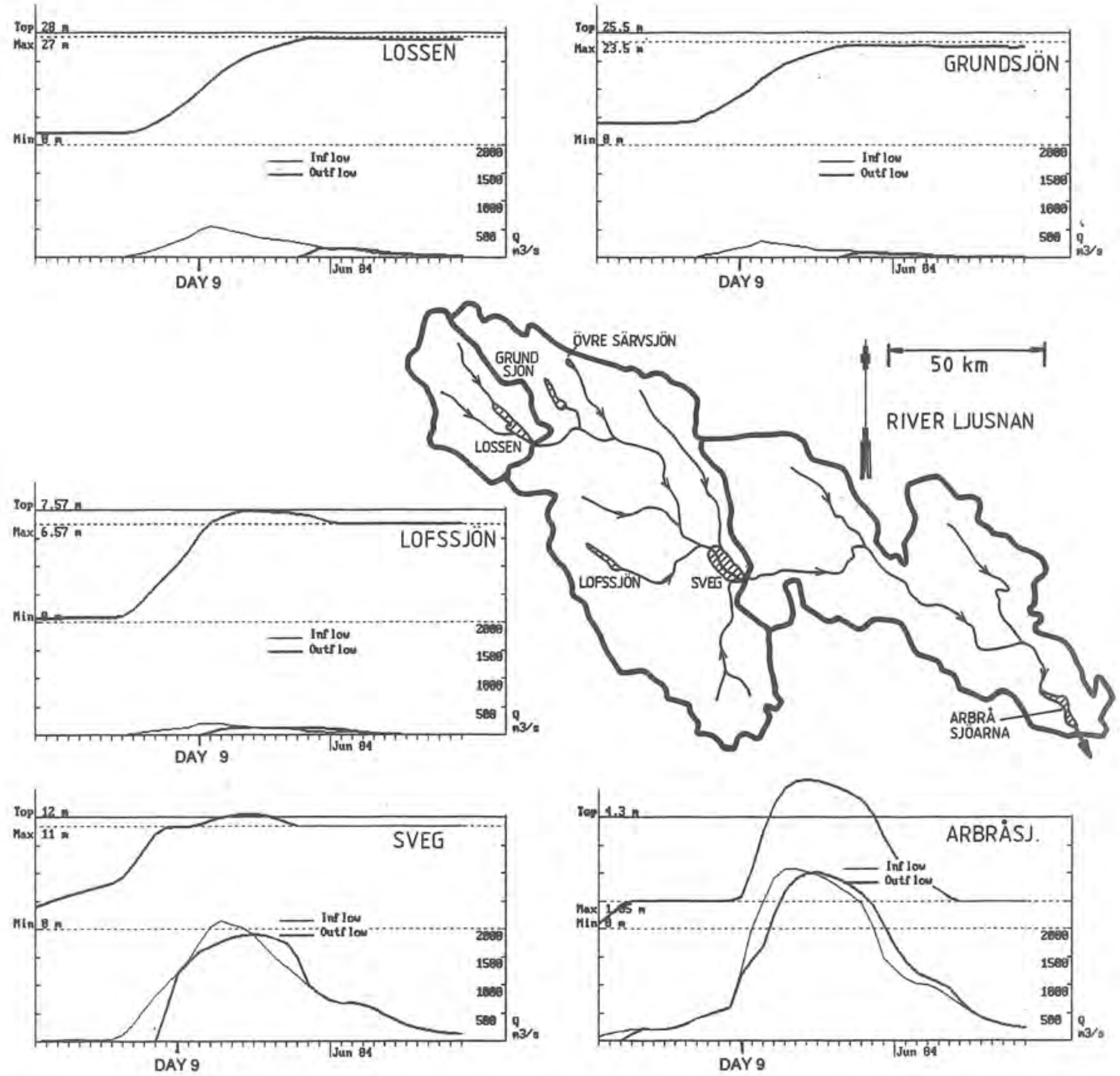


Figure 1. *The river Ljusnan basin in central Sweden, and the basin subdivision for the studied reservoirs. The water levels, inflow and outflow hydrographs refer to the design situation at Arbråsjöarna. This timing of events was also the most critical for the Sveg reservoir. Top = top of dam, Max = maximum pool elevation, Min = minimum pool elevation, and DAY 9 refers to the occurrence of the peak in the design rainfall sequence.*

The total drainage area at the outlet in the Baltic sea is 19 816 km^2. Most of the basin is covered by coniferous forest and the lake percentage is 4 %. The basin receives an annual average rainfall of 900 mm in the western mountain range, and 650 mm in the eastern part of the basin. The annual average evapotranspiration varies from 200 mm in the west to 400 mm in the east. The unregulated flow in the river has a clear seasonal pattern, with large snowmelt floods in spring (April and May), occasional rain floods in summer and autumn (June to November), and base flow during winter (December to March). The power production in the River Ljusnan amounts to 3.9 TWh in a normal year, and the installed capacity is about 750 MW. Key data on the three basins and reservoirs in the study are given in Table 1.

Table 1. Key data on basins and reservoirs.

Reservoir	Lossen	Sveg	Arbråsjö-arna
Total basin area (km^2)	1 353	8 490	14 560
Local basin area (km^2)	1 353	5 860	6 070
Active reservoir volume (Mm^3)	500	237	40
Min. prescribed release (m^3/s)	3	0	0
Installed turbine capacity (m^3/s)	60	200	300
Spillway capacity at top of pool (m^3/s)	620	1 600	830

Regulation strategies

Tentative regulation strategies were formulated in cooperation with the river regulation enterprise for River Ljusnan. Regulation strategies were formulated for all major reservoirs in the system, and not only the three reservoirs under study. According to the guidelines the reservoirs shall be operated in a way which is normal in a situation with a large snowpack. The occurrence of extreme rainfall is, however, not assumed to be known in advance.

At the start of the design calculations all reservoirs were assumed to be lowered to the mean lowest level at the beginning of the spring flood. These levels were based on about 10 to 20 years of observations. Minimum release requirements were followed, until the water level reached a certain level, above which the power production was started, and the spillways were gradually opened. This level was in most reservoirs set equal to one meter below the maximum pool elevation. The spillways were assumed to be fully opened at the maximum pool elevation. If the water level continued to rise, due to extreme inflow, the release was calculated according to the rating curve for the fully opened spillways.

In accordance with the guidelines, the transmission from the power plants was assumed to fail on the day with most intense rainfall, day nine in the design sequence. The release from the turbines was therefore stopped on the beginning of this day. Power production was assumed during summer and autumn, but the dams were filled to maximum pool elevation no later than 1 August. After this level was reached, the water levels were not lowered below it again. In this text all events before 1 August are referred to as spring events, and all events after this date are referred to as autumn events.

The top of dam levels are the actual crest for some reservoirs and the top of the impervious core in others. This level is primarily intended as a reference, and should not be interpreted as a level causing dam failure. In some simulations the water levels reached above the top of the dam. All calculations were therefore made with the assumption that these reservoirs had been raised in order to endure the high water levels. Furthermore, the possible existence of emergency spillways (fuse-plugs etc.) was not taken into account in this study. The results should thus be interpreted qualitatively rather than quantitatively.

The hydrological model

All design calculations according to the guidelines were made using a model system based on the HBV conceptual runoff model (Bergström, 1976). A brief description of the basic runoff model is given in the companion paper by Harlin and Lindström (1992). For routing through reservoirs, as in this study, the model is complemented with level pool routing of inflow hydrographs through reservoirs according to the given operation strategies.

The HBV model was set up and calibrated for 5 points in the river Ljusnan basin: the inflows to Lossen, Grundsjön, Lofssjön, unregulated local inflow to Sveg, and unregulated local inflow to Arbråsjöarna. The total basin was divided into 16 subbasins. The model was calibrated manually, with emphasis on the two peak flow parameters K_0 and MAXBAS. K_0 determines the recession rate for the upper part of a flood hydrograph. MAXBAS is a routing parameter within a subbasin. When linking the subbasins, a time lag (BLAG), corresponding to the flow time between subbasins, was used. The flow time from Lossen to the Sveg reservoir, for example, is three days. The chosen parameter values are given in Table 2.

Table 2. Analysed HBV Model parameter values.

Parameter	Lossen	Sveg	Arbrå-sjöarna
K_0 (day^{-1})	0.25	0.25	0.16
MAXBAS (days)	2	3	3
Travel time BLAG to downstream reservoirs (days)	3	1	-

Sensitivity analysis procedure

Design inflows and water levels were first calculated strictly according to the guidelines, giving reference values. New design inflows and water levels were then calculated, by changing one factor at a time while keeping all others as in the reference run. The sensitivity of design water levels was studied with respect to changes in guideline prescriptions, reservoir operation strategies and model parameters.

Design water stages were calculated with changes of +-10 % and +-20 % in the design precipitation, in the design snow pack, in the model parameter K_0 and in the spillway capacity. MAXBAS was changed in steps of 1 day. K_0 and MAXBAS were changed in the local basins used in the calibration, while keeping the upstream basins at reference values. The spillway capacity was only changed in one reservoir at a time. The design precipitation and snow packs were, on the other hand, changed for the total river basin. The importance of the regulation strategy was investigated by changing initial water levels, buffer storage and release strategies. The initial water levels were varied between the lowest and highest observed levels at the beginning of the spring flood.

To investigate the importance of timing in the basin, the travel times BLAG were set equal to zero. Finally, the design precipitation sequence was routed downstream in the basin, so that the movement of the design precipitation coincided with the flood hydrograph.

RESULTS AND DISCUSSION

Figure 2 shows the design water stage for alternative timings of the design precipitation sequence. Only a short period resulted in critical water levels for the Sveg reservoir, whereas almost one month was critical for Arbråsjöarna. The same periods were the most severe in Sveg and Arbråsjöarna, whereas the most severe spring events occurred later in the year in Lossen. This is due to the later snow melt at higher elevations and a large flood moderating reservoir volume. In fact, higher water levels in Lossen occurred in autumn than in spring, whereas for the other two reservoirs the spring was much more critical. Figure 2 also indicates that the ability of the reservoirs to handle the design inflow decreased downstream in the River Ljusnan, although the design precipitation intensities decrease with increasing basin area.

Strong relations were found between the highest inflow peaks and the highest water levels in Sveg (Figure 3) and Arbråsjöarna. In Lossen the large reservoir capacity distorted the relation considerably. The highest inflow peaks were there caused by combined snow melt and rainfall in the spring, when the reservoir had yet not been filled. However, after the filling of the reservoir, there was a clear relation between peak inflow and highest water stage also in Lossen.

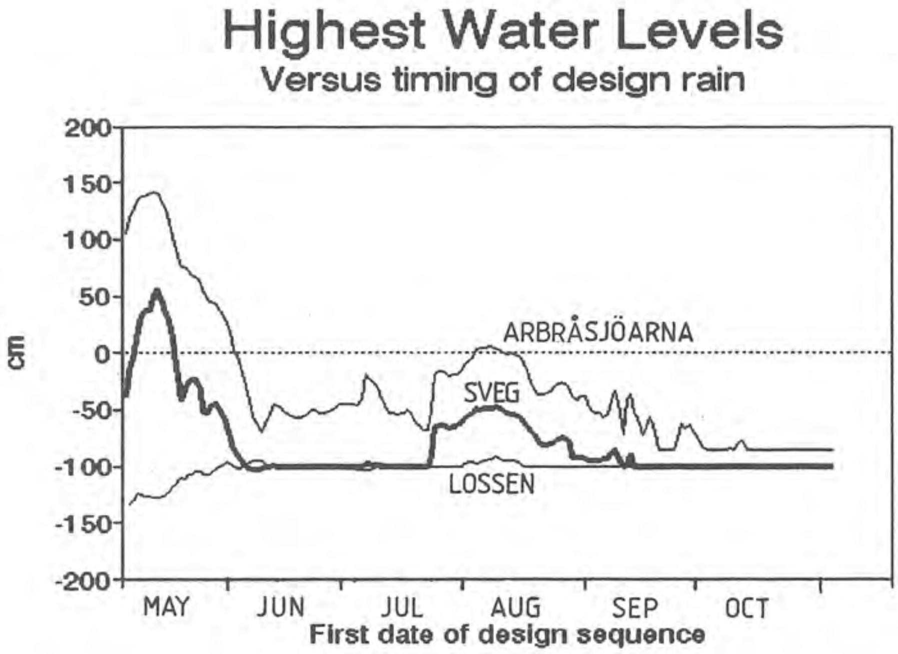


Figure 2. The highest water levels versus timing of design rainfall over a ten year simulation period.

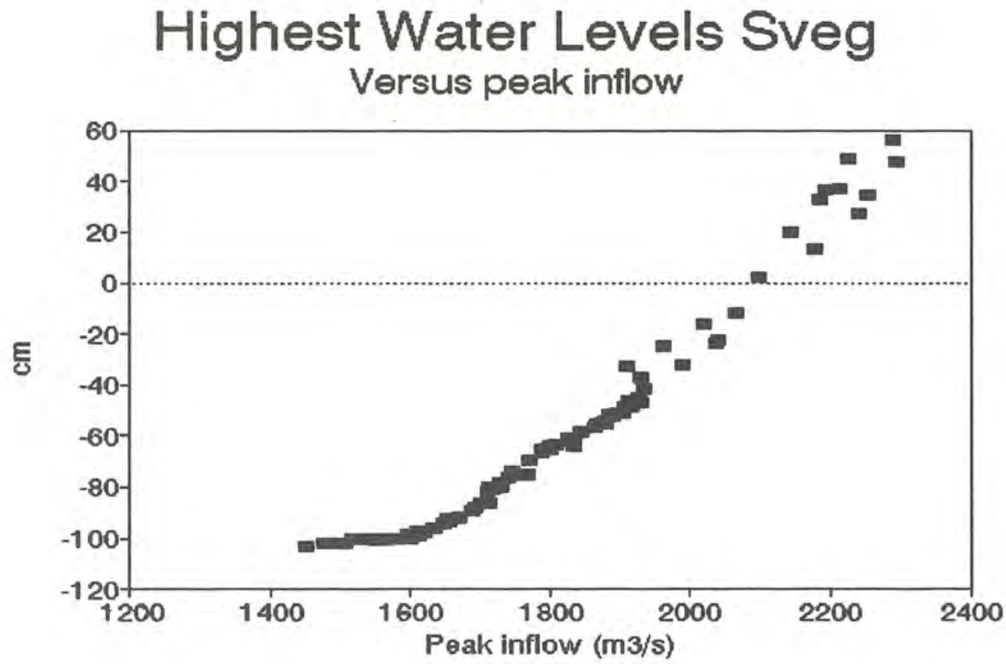


Figure 3. Inflow peaks in relation to highest water levels for Sveg.

Reference simulations

The highest water stage in Lossen was obtained when the design precipitation was introduced into the observed data from august 1989, but the level was only a few cm higher than the worst spring situation (Figure 4).

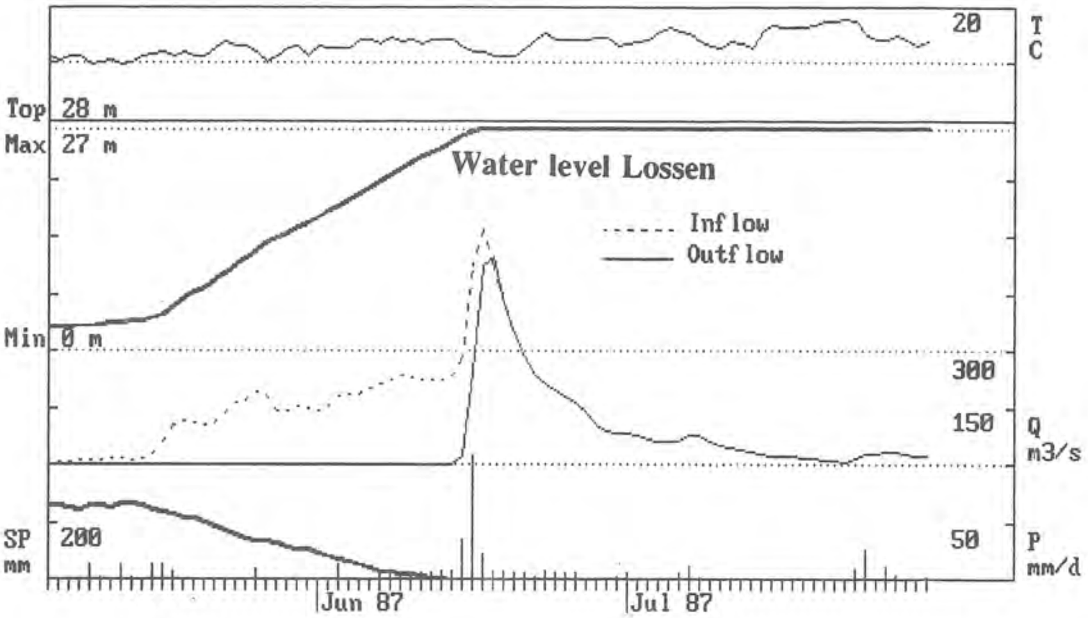


Figure 4. *The most critical spring situation for Lossen. Top = top of dam, Max = maximum pool elevation, Min = minimum pool elevation, SP = snow pack, T = temperature and P = precipitation.*

In Sveg the design spring flood was much more severe than the autumn flood. In fact, with the spillway capacity used in the calculations, the reservoir was not able to handle the spring conditions (Figure 1), whereas it could stand the most critical autumn conditions. The most critical situation in Sveg occurred so early that the large reservoirs Lossen and Grundsjön had not yet been filled up. From the upstream reservoirs only small contributions took place from Lofssjön and Övre Särvsjön. Even without any release at all from these two upstream reservoirs, the water levels reached almost as high as in the reference run. It is therefore not possible to significantly improve the situation at Sveg by upstream reservoir operation. The contribution from snowmelt in the spring thus dominates over the contribution from upstream reservoirs in the autumn, despite the larger design precipitation in autumn. The local design precipitation over the Sveg basin only, produced design water levels only 2 cm lower than in the total design, because of the very small contribution from the upstream reservoirs.

As in the simulation for Sveg, the most critical situations in Arbråsjöarna occurred in the spring (Figure 1). The difficulty in Arbråsjöarna was to a large extent caused by release from Sveg, which contributed to more than half of the peak inflow to Arbråsjöarna. Without the release from Sveg, the water level in Arbråsjöarna did not reach the dam top, nor did the local design simulation.

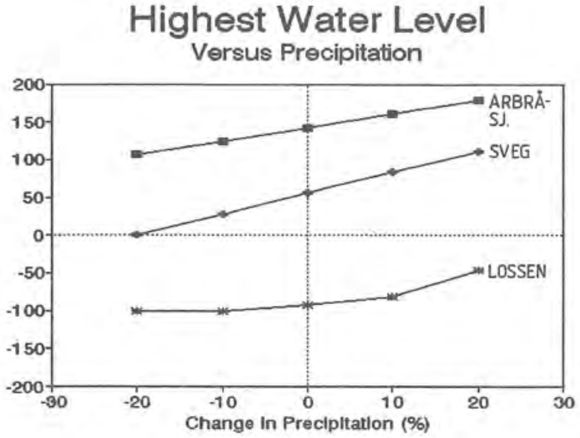
Sensitivity analysis

The timing that resulted in the most critical events in the reference run was usually the most severe also after the changes made in the sensitivity analysis. However, this was not always the case. In Lossen, for example, some changes resulted in a new critical timing, so that the spring flood was more critical than the autumn flood.

Alterations in the preset design precipitation and snow pack had a significant impact on the highest water stage in all three reservoirs (Figure 5). The changes of the reservoir operation strategies were less important (Table 3). The initial water levels in Sveg and Arbråsjöarna had no effect at all on the design water stage. The initial levels in the upstream reservoirs, had very small influence on the downstream reservoirs, when changed within observed levels (Table 3). Much more crucial for the water stage development was the spillway capacity (Figure 6). This is also illustrated by the importance of the prescribed loss of spillway capacity through the turbines on day nine in the design precipitation sequence (Table 3). Increased spillway capacity, on the other hand, worsens the situation downstream (Table 3).

Table 3. *Results from the sensitivity analysis on regulation strategies. 1) No release in reference run. 2) No upstream reservoirs.*

Change to reference	Design water stages (cm above top of dam)		
	Lossen	Sveg	Arbråsjöarna
Reference	-92	+56	+142
Zero release up to top of pool in studied reservoir	-84	+57	+142 1)
Increased spillway capacity to eliminate storage above top of pool in upstream reservoirs	2)	+91	+224
No failure in power production during the storm in studied reservoir	-99	± 0	+ 90
Initial water stages in all reservoirs at lowest recorded levels	-92	+55	+142
Initial water stages in upstream reservoirs at highest recorded levels	-92	+56	+144
Gradual release in studied reservoir starts 1 m lower than in reference run	-95	+56	+142
No time lag (BLAG) from upstream reservoirs	2)	+79	+164



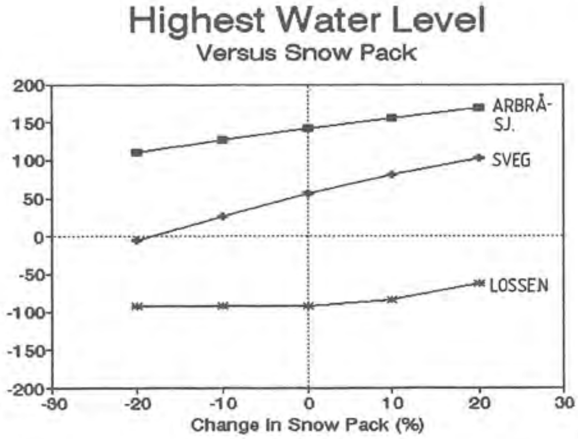


Figure 5. *Effect of changes in the design precipitation and the design snow pack on the highest water level.*

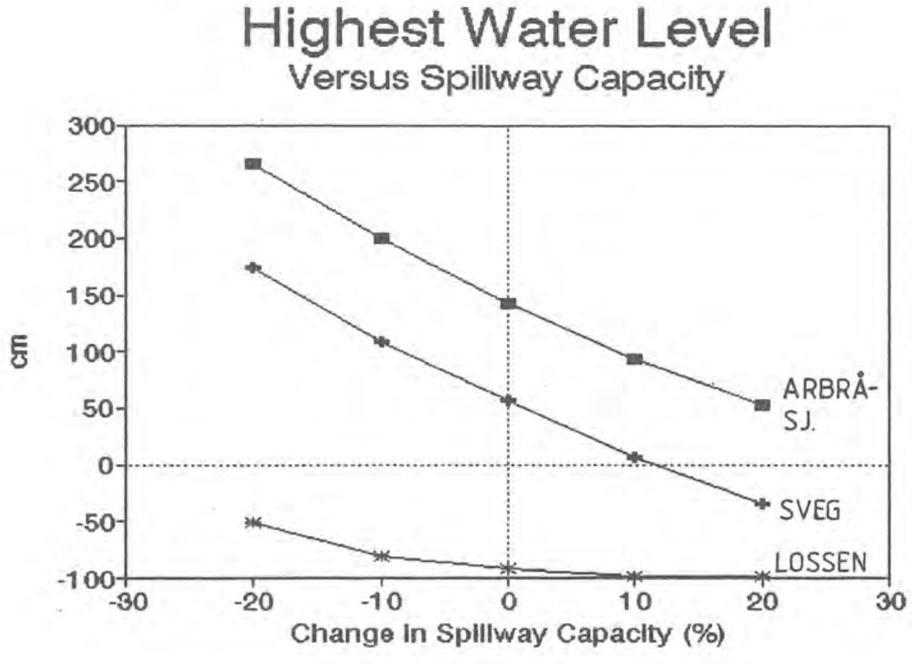


Figure 6. *Effect of a change in the spillway capacity on the highest water level.*

The design water stage was sensitive to the HBV model parameter K_0 (Figure 7). The model structure assumes a linear extrapolation to extreme floods by this parameter. A change in K_0 can therefore also illustrate the effects of any possible non-linearity not accounted for by the model structure.

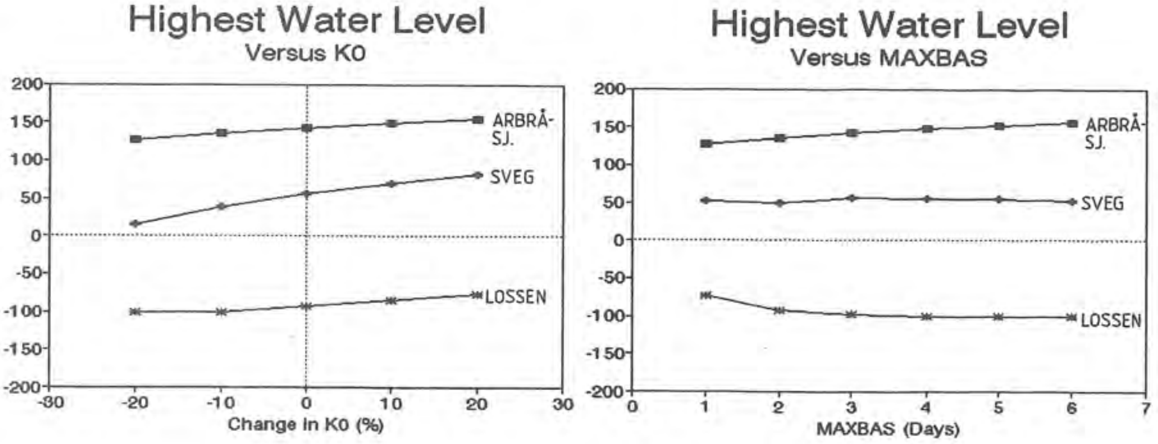


Figure 7. Effect of changes in K_0 and MAXBAS on the highest water level.

The routing in the basin, represented by the model parameter MAXBAS (Figure 7), was less important. An increase in MAXBAS in a downstream subbasin could even result in more critical situations by superposition with release from upstream reservoirs. It did, however, affect the simulation for Lossen, and has importance in head water basins. The time lag, represented by the model parameter BLAG, was found to be slightly more sensitive. The effect of moving the design precipitation downstream, coinciding with the flood hydrograph, was approximately the same as when the time lag (BLAG) was disregarded.

CONCLUSIONS

The design calculations for River Ljusnan show that it is difficult to before hand assess the integrated effects of extreme precipitation, snow melt, soil moisture status and reservoir operation in a system. A hydrological model should be used in a trial and error procedure to find the timing that gives the most critical water stage development. This timing of events was usually the most critical one also after moderate changes in the design assumptions. This fact can be used to save time in practical design computations. After the filling of a reservoir, there was a clear relation between the inflow peak and the highest water stage. This means that knowledge about how different factors influence the peak inflow, as studied in the companion paper by Harlin and Lindström (1992), can also give information on the effect on water stage development.

The design water stage depends on a number of factors, of which the preset design precipitation and snow pack are the most important. The spillway capacity is of course crucial, and a small increase in spillway capacity can improve the situation for a particular dam considerably. The chosen regulation strategy and initial water levels appear to be less important, when varied within reasonable limits.

In the application of the spillway design guidelines to a system of reservoirs, careful model calibration is important, with the highest recession coefficient K_0 as the most important parameter. The importance of this parameter is not only restricted to the upper parts of the river. It was found to be much more important than the routing parameter MAXBAS. An increase in MAXBAS, due to storage in the basin, can even result in a more critical situation, because of the timing with release from upstream reservoirs.

The design water stages were sensitive to changes in some of the design factors. However, these factors could be varied considerably without changing the conclusion on whether a particular dam could stand the new guidelines or not.

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IV

Development of a process-oriented calibration scheme for the HBV hydrological model

by

J. Harlin

Nordic Hydrology, 22, 1991, 15-36

Development of a Process Oriented Calibration Scheme for the HBV Hydrological Model

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A process oriented calibration scheme (POC), developed for the HBV hydrological model is presented. Twelve parameters were calibrated in two steps. Firstly, initial parameter estimates were made from recession analysis of observed runoff. Secondly, the parameters were calibrated individually in an iteration loop starting with the snow routine, over the soil routine and finally the runoff-response function. This was done by minimizing different objective functions for different parameters and only over subperiods where the parameters were active. Approximately three hundred and fifty objective function evaluations were needed to find the optimal parameter set, which resulted in a computer time of about 17 hours on a 386 processor PC for a ten-year calibration period. Experiments were also performed with fine tuning as well as direct search of the response surface, where the parameters were allowed to change simultaneously. A calibration period length of between two and six years was found sufficient to find optimal parameters in the test basins. The POC scheme yielded as good model performance as after a manual calibration.

Introduction

Conceptual hydrological models are becoming increasingly used tools in solving practical water resources engineering problems. Advances in computer facilities have enhanced this development and today many models are run on personal computers. Commonly, hydrological models are used for forecasting and for hydrologic design. In Scandinavia for instance, the HBV model (Bergström 1976) is widely spread and has for the last decades been operationally used for flood warning and inflow forecasting to hydro-power reservoirs.

Currently new spillway design guidelines are being adopted in Sweden (Swedish Committee on Spillway Design 1990). The design floods are generated using a hydrological model. Bergström, Lindström and Sanner (1989) and Harlin (1989) describe the methodology and discuss the return periods of the spillway design floods. However, the question of how the model calibration affects the floods has not yet been addressed.

The accuracy of a model output is dependent on the quality of the input data, the model structure and the calibration. In the HBV model, as in all hydrological models, a number of parameters are not directly measurable and have hence to be calibrated. Calibration can be formulated as: to obtain a unique and conceptually realistic parameter set so that the model becomes specific to the system it simulates and performs well. Manual calibration is often a tedious trial and error procedure, whereby the parameters are adjusted by matching the input/output behaviour of the watershed to that of the model. To calibrate the HBV model requires a thorough understanding of the model structure and experience of how the parameters should be changed to achieve an optimal performance. The quality of a manual calibration is often a function of the users knowledge and the time spent calibrating the model.

Unfortunately there has been limited success achieved in relating the parameters of hydrological models to catchment characteristics. Another problem is the lack of representative areal input data. It is also desirable to restrict the number of parameters in a model in order to reduce the data demand and risk of overparameterization. This leads to simplificatons in the description of physical processes and introduces unmeasurable calibration parameters. Even apparently measurable parameters of more complex models often devolve to calibration parameters. It was, for example, found necessary to calibrate the saturated permeability in the ILWAS model when applying it to the Woods Lake catchment in New York, USA (Chen *et al.* 1982). Also in as complex formulations as the SHE model there are inevitably approximations in the representation of the physical processes which lead to calibration parameters (Bathurst 1986). Therefore operational hydrology will have to rely on either manual or automated search techniques for model calibration.

When calibration is done by an automatic algorithm the problem can be formulated as to find those parameter values that maximize or minimize an objective function $OF = f(p1, p2, p3, \ldots, pn)$, where $p1, \ldots, pn$ are the model parameters. OF is a measure of how closely the model-computed runoff compares with the runoff actually measured. In automatic calibration the computational effort is dominated by the cost of evaluating OF for new parameter values, *i.e.* a new model run. Therefore, the strategy is to find the optimum, evalutating OF as few times as possible.

This paper describes a process oriented automatic calibration scheme (POC), developed for the HBV hydrological model. It has been developed using real data on a daily time step. Firstly, a literature review of calibration strategies and commonly experienced problems is given. Secondly, the HBV model and the calibrated parameters are described. Finally, the calibration scheme is presented followed by a discussion of its performance.

Literature Review

Automatic calibration approaches have been extensively discussed in literature. Most commonly the runoff response equations have been studied, but in some cases soil moisture equations have also been included. A popular calibration approach is to use direct search techniques, such as the downhill simplex method (Nelder and Mead 1965), Rosenbrock's coordinate rotation method (Rosenbrock 1960) or Powell's conjugate directions method (Powell 1964).

Ibbitt and O'Donnell (1971) presented a comparison of nine different optimization methods based on experiments performed on the Dawdy and O'Donnell model. They concluded that the decision of the best method depended on what criterion of goodness was used. They selected the Rosenbrock method modified by Ibbitt (1970) as the most efficient one. Johnston and Pilgrim (1976) used the simplex and Fletcher-Powells descent methods in a detailed calibration scheme for the Boughton model. Improvements in the calibration procedure were achieved by modifying the search methods so that the model characteristics were accounted for. Their main problems were: 1) interdependence between model parameters, 2) indifference of the objective function to parameter changes, 3) discontinuities and local optima on the response surface and 4) the ability of the optimization methods to adjust to the response surface being searched. Similar problems were also experienced by Pickup (1977), who tested the efficiency of several calibration algorithms on the Boughton model.

Another calibration strategy is based on trial and error schemes. Sugawara (1979) reported the application of an automatic trial and error calibration method for the Tank model. He used a feedback procedure that evaluated the model performance and divided the total period into subperiods. The subperiods where selected so that the output during each period was governed by one tank. He used volume and shape criteria to adjust the parameters and claimed a high rate of convergence.

Many researchers have developed stochastic estimation procedures, for example Restrepo-Posada (1982), who worked with a simplified version of the NWSRFS model. He pointed out the importance of restricting the parameter number and suggested a modification of the upper zone tension water element so as to make it permanently observable as in the HBV model. Comprehensive studies within this field have also been made by Sorooshian and Dracup (1980), Sorooshian and

Gupta (1983), Gupta and Sorooshian (1985) among others.

Brazil (1989) suggested a three level approach for calibrating the NWS Sacramento Soil Moisture Model. Level one was a guided interactive initial parameter estimator. Level two was an adaptive random search of the parameter space isolating the global optimum. Level three was a fine tuning of the parameters by a pattern search method or a Kalman filtering procedure. He concluded that for most purposes a final calibration result was produced already after level two.

Attempts to automate calibration of the HBV model have also been made. Bergström (1976) used Rosenbrocks method which proved to be able to fit the model rapidly, but he reported on several restrictions which "gave rise to more scepticism than enthusiasm". He listed the following difficulties: choice of objective function, lack of information of the response surface topography and convergence to unrealistic parameters or local optima. Svensson (1977) examined the statistical properties of the residuals of the HBV model in order to guide an automated calibration strategy. He found that the residuals were neither stationary nor independent or normally distributed. After separating the residuals in sets governed by separate processes the autocorrelation was reduced and the residuals became more stationary distributed. These findings together with the long experience of manual calibration at the Swedish Meteorological and Hydrological Institute (SMHI), have formed the base for the calibration methodeology presented in this paper.

Model Structure and Calibrated Parameters

The HBV model was originally developed for use in Scandinavien catchments but has proved to run well in tropical and sub-tropical areas as well, see for example Häggström *et al.* (1990) and Bathia *et al.* (1984). For most applications, the model is run on daily values of rainfall and temperature and monthly estimates of potential evapotranspiration. It consists of routines for snow accumulation and melt, soil moisture accounting, runoff response and, finally, a routing procedure. The model can be used in a distributed mode by dividing the catchment into subbasins. Each subbasin is then divided into zones according to altitude, lake area and vegetation. The snowroutine is based on a degree-day approach and runs separately for each elevation and vegetation zone according to the equation

$$d\mathrm{Melt} = CF\mathrm{MAX}(T-TT) \tag{1}$$

where *d*Melt is the snowmelt per timestep, *CF*MAX is the degree-day factor, *T* is mean air temperature and *TT* is the threshold temperature for snowmelt and snow accumulation. There is also a general snowfall correction factor (*SFCF*) which adjusts systematic errors in calculated snowfall.

Parameters that were calibrated from the snow routine were *SFCF*, *CF*MAX

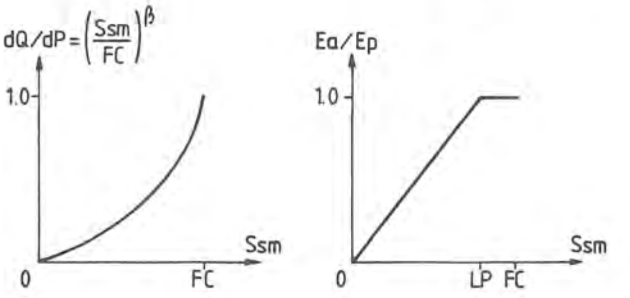

Fig. 1.
Schematic presentation of the soil moisture and evapotranspiration relations in the HBV model.

and *TT*. *TT* was set equal in all vegetation zones, and fixed relations between forest and open area for *SFCF* and *CFMAX* were used.

The soil moisture routine is based on three empirical parameters: β, *FC*, and *LP*, as shown in Fig. 1. β controls the contribution (*dQ*) to the runoff response routine and the increase (1-*dQ*) in soil moisture storage (*Ssm*). *FC* is the maximum soil moisture storage in the model and *LP* is the value of soil moisture, above which evapotranspiration (*Ea*) reaches its potential level (*Ep*). Since mass balance over the soil states: $dQ = dP - dSsm$, soil moisture accounting can be expressed as

$$\frac{dSsm}{dP} = 1 - \left(\frac{Ssm}{FC}\right)^{\beta} \qquad (2)$$

The parameters *FC*, *LP* and β were included in the calibration.

Excess water from the soil is transformed by the runoff-response function. This routine consists of two tanks which distribute the generated runoff in time, so that the quick and the slow parts of the recession are obtained (Fig. 2). The lower tank is a simple linear reservoir representing the contribution to base flow. It also includes the effects of direct precipitation and evaporation over open water bodies in the basin. The lower tank storage (*Slz*) is filled by percolation from the upper tank (*PERC*), and K_2, is the recession coefficient.

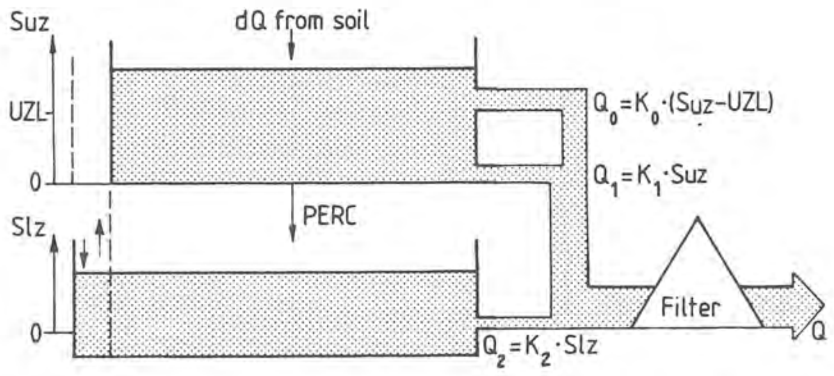

Fig. 2. The runoff-response function of the HBV model.

If the yield (dQ) from the soil moisture routine exceeds the percolation capacity, the upper tank will start to fill. Upper tank storage (Suz) is drained by two recession coefficients K_0 and K_1, separated by a threshold (UZL). This tank models the response at flood periods. Parameters calibrated from the runoff-response function were $PERC$, K_2, K_0, K_1 and UZL.

Finally, runoff is computed independently for each subbasin by adding the contribution from the upper and the lower tank. In order to account for the damping of the flood pulse in the river before reaching the basin outlet, a simple routing transformation is made. This filter has a triangular distribution of weights with the base length MAX*BAS*. Including MAX*BAS* the total number of calibrated parameters amounted to twelve.

Philosophy of the Process Oriented Calibration Scheme (POC)

The philosophy of the calibration scheme has been to utilize the physical representation of the model components and the experience from manual calibrations. This was done by splitting the calibration period into subperiods, within which one specific process dominates the runoff. In this manner the parameters are only evaluated over the subperiods where they are active (Fig. 3). The physical representation of the model components is known, and therefore the calibration should optimize them only when the physical processes they resemble are at hand. This is also an important step in order to avoid the effects of parameter interaction. If the objective function is computed for the whole period, this interaction would create noise on the objective function, with respect to the studied parameter.

By splitting the calibration period, different criterions could be used for different parameters. Since the objective functions are computed only over subperiods, where the current parameters are active a clearer picture of the error caused by each one is received. With this strategy, opposed to direct mathematical optimisation, the final calibration result is not a function of a blunt general fit criterion but a result of sub-optimisation of the different runoff generating procedures. This resembles the strategy of a manual calibration. An experienced hydrologist would calibrate the parameters by visual inspection of the model performance over the different hydrograph components and only use the objective function for the whole period as a guidance.

The different subperiods were compiled by combining the observed temperature and the observed runoff data. This program was made interactive so that the user had the possibility of adjusting the computer-suggested periods. From the duration curve of observed runoff, characteristic high flow (Qh) and baseflow (Qb) discharge limits were estimated. A discharge larger than Qh was regarded as a flood, and a discharge lower than Qb formed base flow. Qh and Qb were found by analyzing the change of slope of the duration curve as illustrated in Fig. 4.

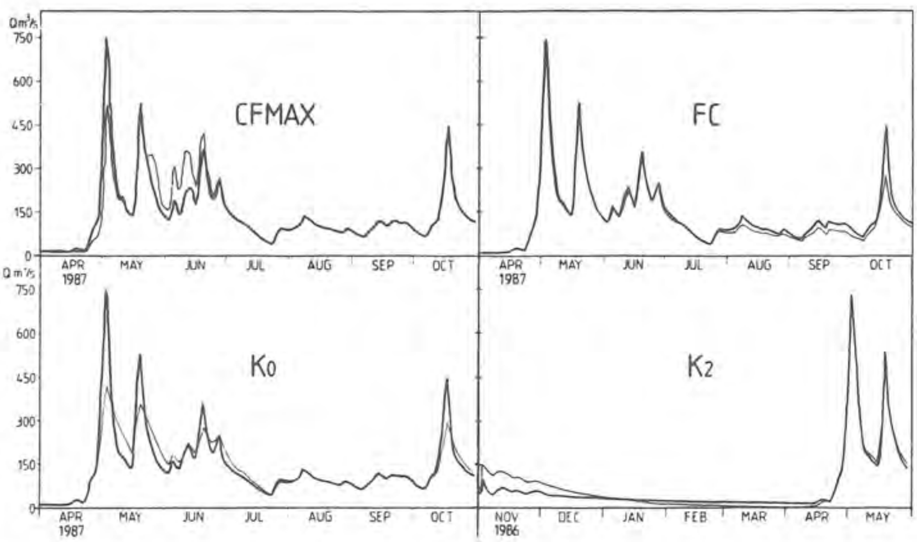

Fig. 3. Example of the active period for the degree-day factor *CFMAX*, the maximum soil moisture storage *FC* and the recession coefficients K_0 and K_2. Thick and thin hydrographs illustrate the effects of changes of the parameter values.

Subperiods dominated by snowmelt floods were found by checking the runoff after cold spells and using *Qb* and *Qh* to follow the floods and define the start and end of them. Warm periods during which runoff was above *Qb* formed the rain flood subperiods. Subperiods dominated by baseflow were compiled by checking when the observed discharge was below *Qb* and so on. Fig. 5 shows an example of how the different subperiods were discriminated.

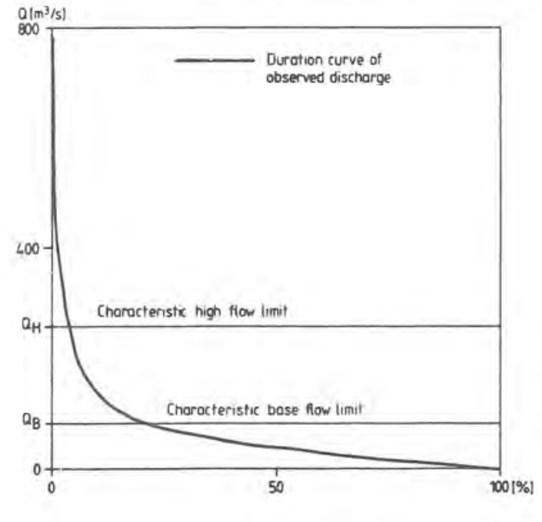

Fig. 4.
Estimation of characteristic high flow and base flow discharge limits from the duration curve.

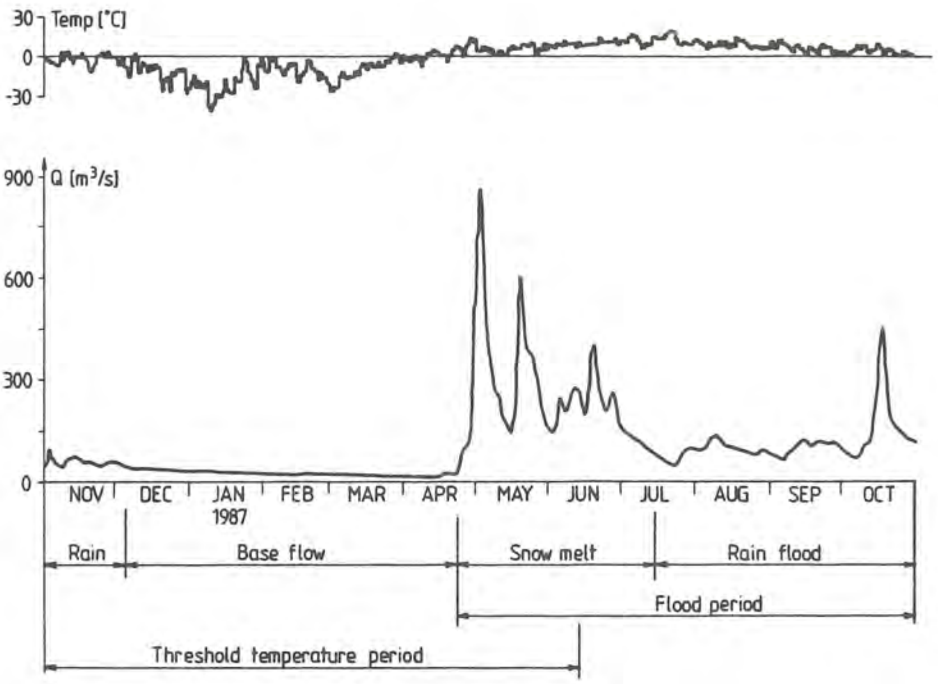

Fig. 5. Splitting of the calibration period in subperiods dominated by one process.

Description of the Process Oriented Calibration Scheme

Step 1 – Initial Parameter Estimation

Initial guesses of the coefficients K_0, K_1 and K_2 were made by recession analysis of the observed runoff larger than Qh, between Qh and Qb and below Qb, respectively, by Eq. (3).

$$K = \frac{1}{dt} (\ln Q_t - \ln Q_{t+1}) \tag{3}$$

Q_t and Q_{t+1} are discharge values at the time dt apart.

The threshold UZL of the upper tank in the runoff-response function was also estimated by converting the characteristic high flow discharge Qh into a storage level by

$$UZL = \frac{Qh\ 86{,}400}{\text{Area}} \tag{4}$$

where Qh is given in mm^3/s, Area is the catchment area in mm^2 and 86,400 is the number of seconds in 24 hours.

An initial guess of the percolation rate $PERC$ could be computed by the same Eq. (4), only exchanging Qh with Qb. This is motivated by the continuity equation of the lower tank, *i.e.* $dSlz = PERC - Q$. At the beginning of the base flow period $Q = Qb$ and $dSlz$ is zero, $PERC$ equals then Qb.

MAXBAS was initially set to one day, and the remaining six parameters were initially set to the middle of their respective ranges found by experience from the large number of manual calibrations of Swedish basins.

Step 2 – Iteration Loop and Criteria of Agreement

An iteration loop was performed over the whole model. The parameters were calibrated one at a time in a set order starting with the snow routine, over the soil routine and finally the runoff-response function. For each subperiod an objective function was computed. These were the mean absolute accumulated volume error, define as

$$MAD = \left| \frac{1}{n} \sum_{t=1}^{n} (Qm(t) - Qo(t)) \right| \tag{5}$$

where

- n – number of timesteps
- Qm – computed discharge
- Qo – observed discharge.

This function was used to minimize the volume error of the snowmelt floods. To adjust the phase error of the snowmelt flood start, the mean accumulated absolute error calculated as

$$MABSD = \frac{1}{n} \sum_{t=1}^{n} (|Qm(t) - Qo(t)|) \tag{6}$$

was found most appropriate. Furthermore, the mean square error MSE defined as

$$MSE = \frac{1}{n} \sum_{t=1}^{n} (Qm(t) - Qo(t))^2 \tag{7}$$

and the efficiency criterion R^2 proposed by Nash and Sutcliffe (1970), expressed as

$$R^2 = 1 - \frac{\sum_{t=1}^{n} (Qm(t) - Qo(t))^2}{\sum_{t=1}^{n} (Qo(t) - Qom)^2} \tag{8}$$

where

$$Qom = \frac{1}{n} \sum_{t=1}^{n} Qo(t)$$

were used. MSE and R^2 are equivalent if evaluated over a single period. MSE was used to calibrate parameters active over several subperiods and R^2 was mainly used to evaluate the resulting total model performance.

Normally there were more than one subperiod for each parameter within the calibration period, for example several snowmelt floods. The objective functions were then calculated individually over each subperiod and averaged according to

$$OF = \frac{1}{N} \sum_{i=1}^{N} of_i \tag{9}$$

where

OF – the objective function value for the whole calibration period
N – the number of subperiods
of_i – the objective function value for each individual subperiod.

OF was minimized separately for each parameter with Brents parabolic interpolation method (Brent 1973).

The iteration loop continued until the parameters stabilized, *i.e.* when the R^2 criterion for the whole calibration period stopped changing. Calibration order, objective function and subperiod for each parameter are given in Table 1.

Table 1 – Calibration order, objective functions and subperiods used in the calibration loop

Parameter	Objective function	Subperiods
Snow routine:		
SFCF	MAD	Snowmelt floods
TT	MABSD	Below +2°C*
*CF*MAX	MSE	Snowmelt floods
Transformation function:		
MAX*BAS*	R^2	Whole period
Soil routine:		
FC	MSE	Rain floods
LP	MSE	Rain floods
β	MSE	Rain floods
Upper response tank:		
K_0	MSE	All flood periods
K_1	MSE	All flood periods
UZL	MSE	All flood periods
Lower response tank:		
PERC	MSE	Base flow periods
K_2	MSE	Base flow periods

*) The subperiods for *TT* were all periods where a 14-day moving average of air temperature was below +2°C.

Experiments with Finetuning and Direct Search Methods

Calibration of one parameter at a time has the disadvantage of not taking interdependence between model parameters into explicit consideration. Neither is the information of how parameters, describing the same process, jointly effect the model performance directly utilized. These effects, however, were reduced by calculating the objective function only over subperiods where individual parameters were active and running several loops over all the parameters. The POC yielded parameter values that were considered close to the optimal set. This was checked by a finetuning procedure; a direct search starting from the POC parameters, calibrating several parameters simultaneously.

Powells conjugate directions method (Powell 1964) was chosen since this routine does not require derivatives of the objective function with respect to the parameters and has proved to work well in connection with calibration of hydrological models (Box 1966, Ibbitt and O'Donnell 1971). The method was slightly modified so that the parameter space could be restricted in order to prevent conversion to unrealistic values. Finetuning of all parameters (except MAX*BAS*), simultaneously as well as in subsets for the model routines, was tried.

As an alternative to the POC scheme, experiments with direct search calibration starting from the initial parameter estimates were performed. It was thus possible to compare the accuracy, efficiency and resulting parameter values between the two methods. Direct search calibration was done with Powells method using the mean square error criterion of fit (Eq. (7)).

Results

Calibration Scheme

Fig. 6 gives some key data and examples of the runoff pattern for the three test basins: Torrön, Trängslet and Simlången. These basins represent three different hydrological regimes. Torrön is located in a mountainous region partly above the tree-line. The runoff follows a clear seasonal pattern dominated by large snowmelt floods in spring and rain floods in autumn and sometimes in winter. Trängslet is mainly located in an inland regime and the basin is covered with forest. Also in this basin, runoff has a clear seasonal pattern dominated by large spring floods. The Simlången basin however, belongs to at totally different regime with a humid marine climate producing mainly rain floods. Snowmelt dominated floods seldom occur in this part of Sweden because the winters are shorter and often disturbed by warm periods. Data periods for the test basins were selected so that all changes of the input data stations were avoided.

Model performance in terms of R^2 values, accumulated relative volume errors and volume errors for snowmelt floods only after POC and manual calibration are given in Table 2.

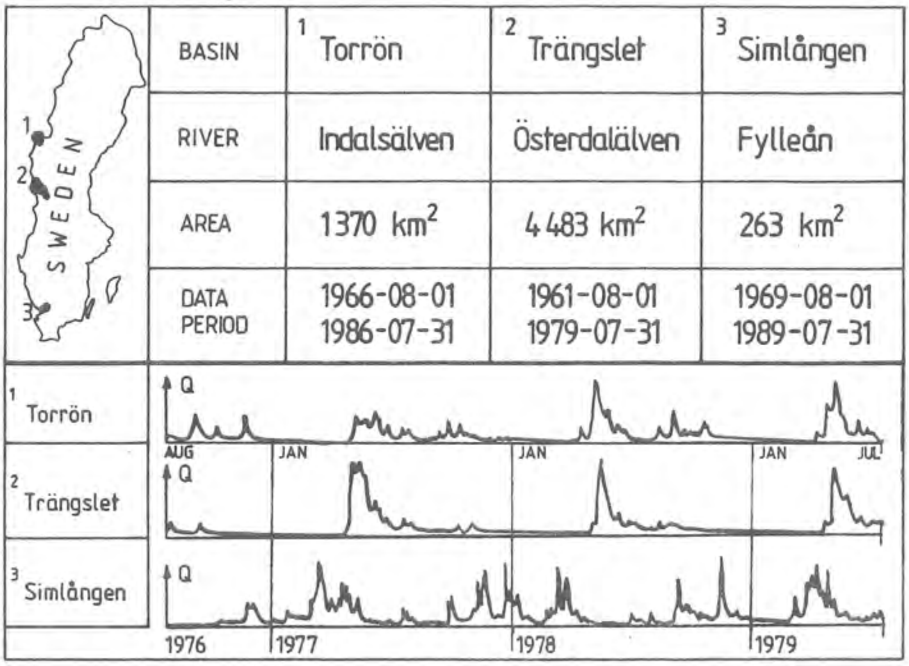

Fig. 6. Key data and examples of the runoff pattern for the test basins.

Table 2 – Model performance after POC and manually calibrated parameters

Basin	Calibration period (POC)	Verification period (POC)	Total period POC	Total period MAN
1. Torrön				
R^2	85.5 (10 years)	84.1 (10 years)	84.8	79.3
VE	0.9	4.2	1.7	3.4
VS	8.0	11.8	10.0	11.8
2. Trängslet				
R^2	94.7 (8 years)	90.7 (10 years)	92.9	92.0
VE	3.5	9.8	4.1	0.8
VS	5.0	10.3	7.8	7.5
3. Simlången				
R^2	89.2 (10 years)	84.3 (10 years)	86.8	83.8
VE	1.9	7.2	4.8	6.6
VS	5.7	11.7	8.7	7.4

R^2 = explained variance (%),
VE = volume error (%), accumulated for all timesteps, and
VS = volume error (%), accumulated over the snowmelt floods.

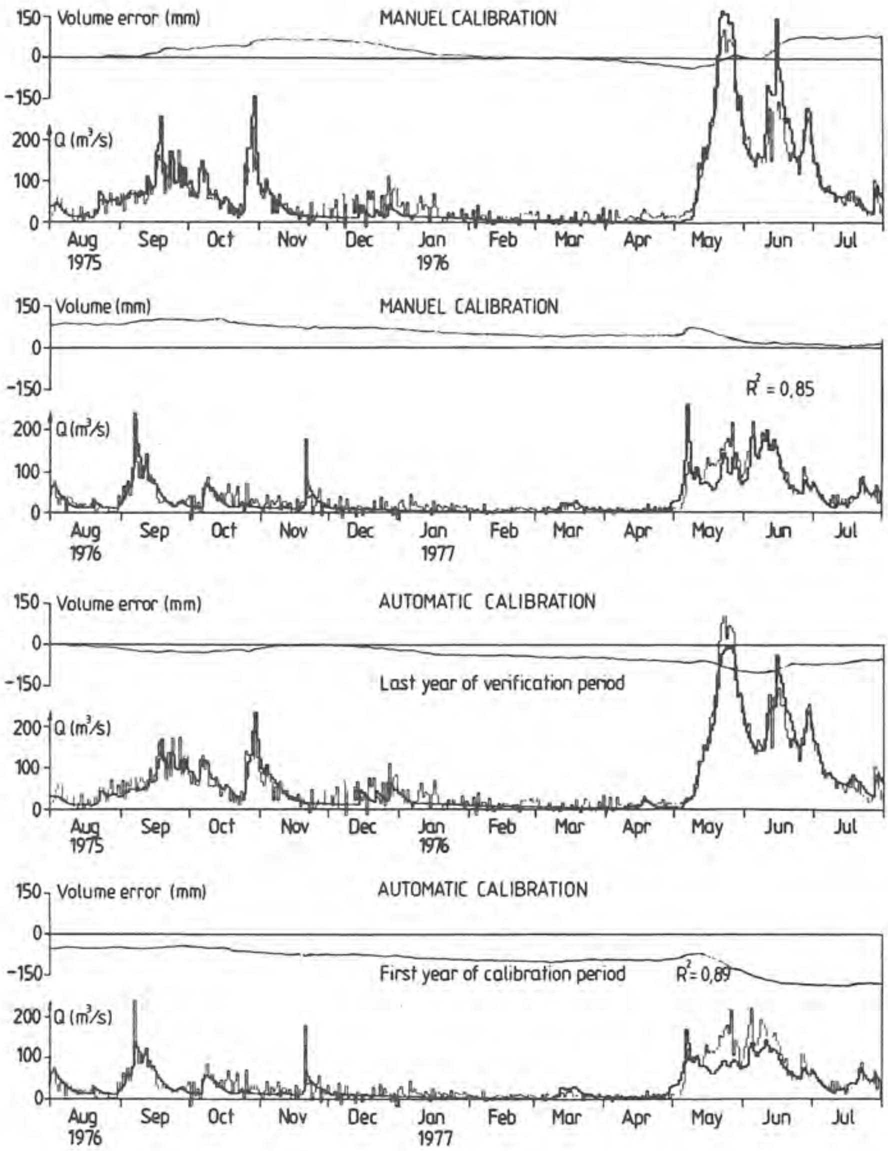

Fig. 7. Example of model performance after manual and automatic calibration (POC), for inflow to the Torrön reservoir. Thick and thin curves show computed and recorded inflow, respectively. The automatic calibration period was 10 years. R^2 values refer to the plotted periods only.

Figs. 7, 8 and 9 show examples of resulting hydrographs after manual and POC calibration. Two hydrological years are depicted for each basin. These figures also illustrate det differences in runoff pattern between the basins.

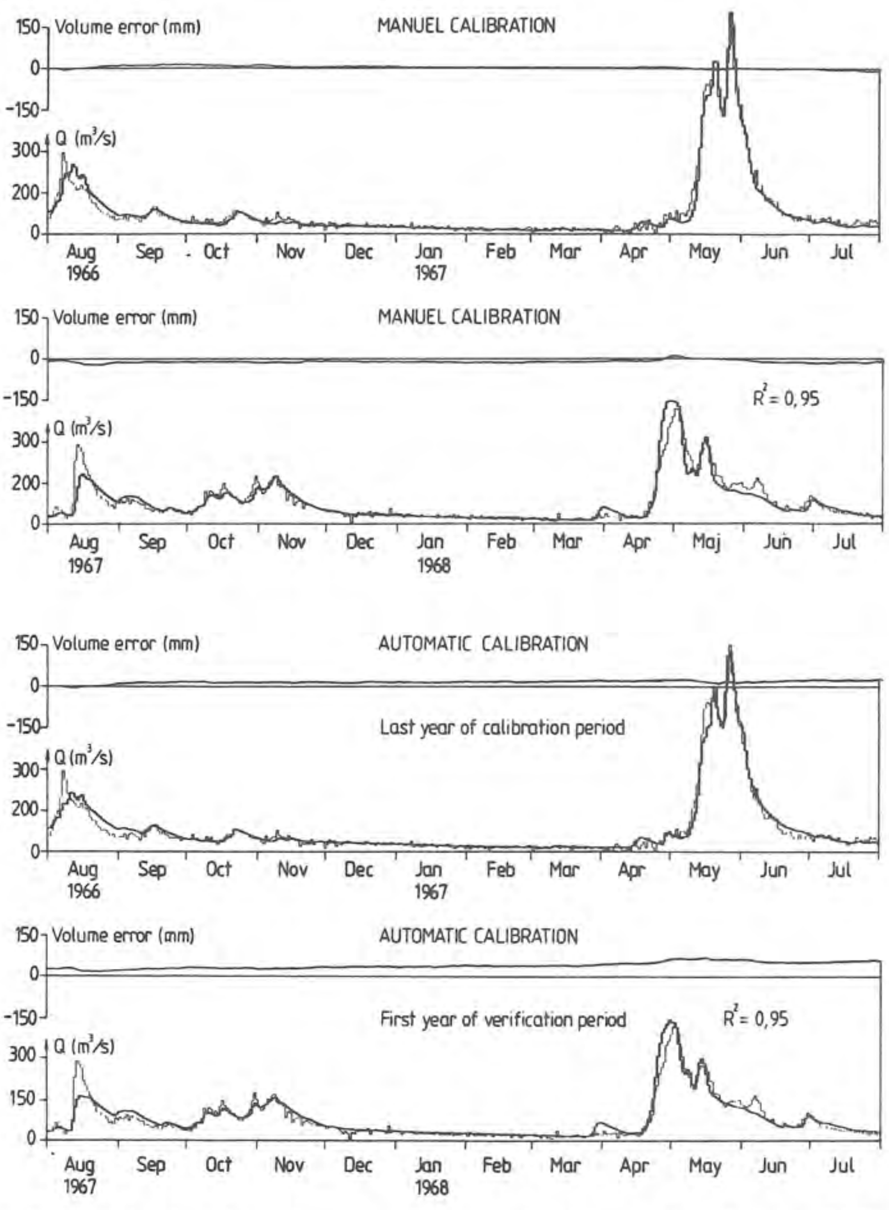

Fig. 8. Example of model performance after manual and automatic calibration (POC), for inflow to the Trängslet reservoir. Thick and thin curves show computed and recorded inflow, respectively. The automatic calibration period was 6 years. R^2 values refer to the plotted periods only.

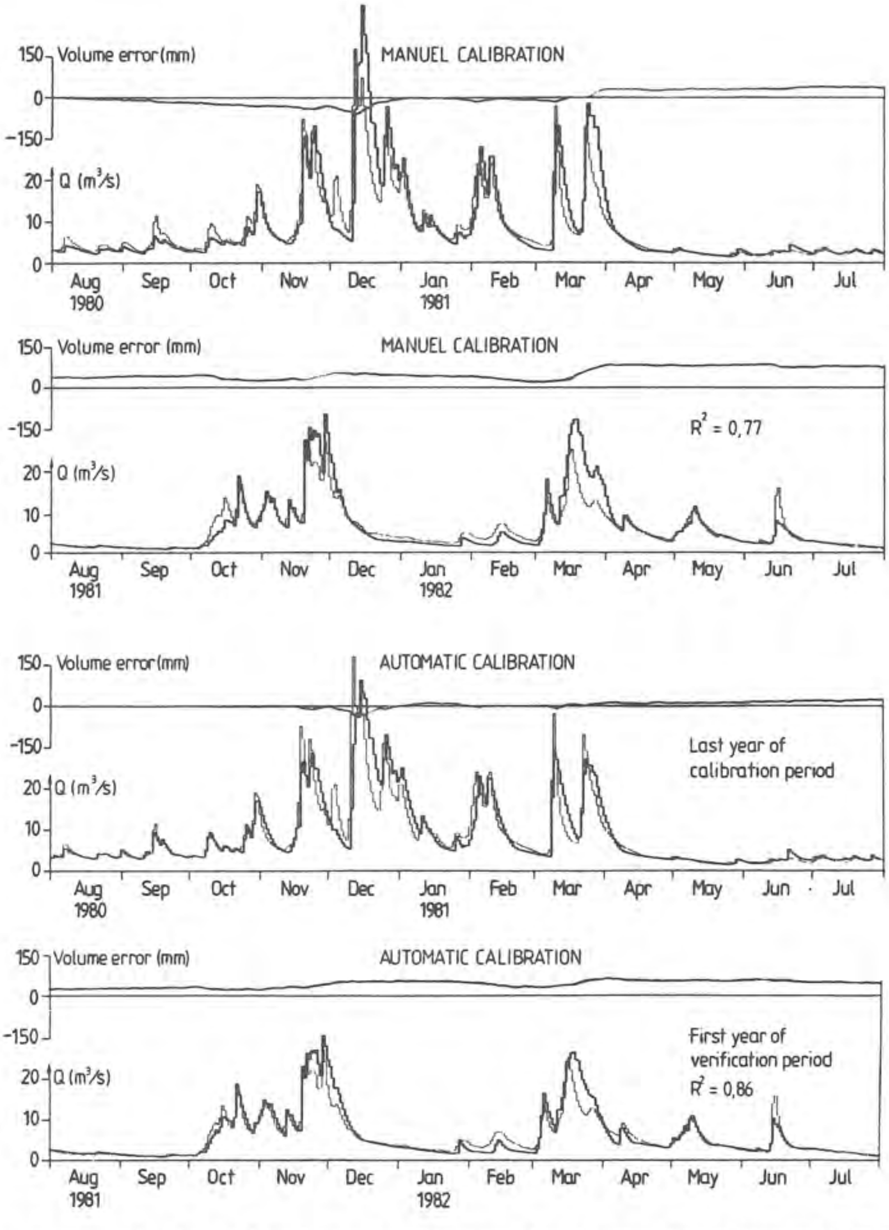

Fig. 9. Example of model performance after manual and automatic calibration (POC), for outflow from lake Simlången. Thick and thin curves show computed and recorded outflow, respectively. The automatic calibration period was 8 years. R^2 values refer to the plotted periods only.

Table 3 – Model performance after calibration by direct search (Powell) and after POC. Performance criteria are compiled for the total data period for each basin; Torrön 20 years, Trängslet 18 years, and Simlången 20 years

Basin	4-year calibration period		6-year calibration period		10-year calibration period	
	Powell	POC	Powell	POC	Powell	POC
1. Torrön						
R^2	83.0	80.0	83.7	83.5	84.0	84.8
VE	2.0	7.3	6.5	3.2	5.0	1.7
VS	9.9	10.4	5.1	12.7	9.8	10.0
2. Trängslet						
R^2	87.2	88.9	89.2	90.9	90.5	92.8
VE	4.8	1.6	5.3	0.4	2.9	4.1
VS	8.1	10.9	7.6	8.6	7.3	7.8
3. Simlången						
R^2	86.9	84.5	89.1	86.3	89.3	86.8
VE	8.0	1.6	0.1	7.7	7.0	4.8
VS	13.2	8.8	6.8	8.2	6.9	8.7

R^2 = explained variance (%),
VE = volume error (%), accumulated for all timesteps, and
VS = volume error (%), accumulated over the snowmelt floods.

Resulting model performances after POC and direct search calibration are given in Table 3.

Computational Speed

Automatic calibration opposed to manual calibration is computer intensive instead of labour intensive. Calibration over a ten-year period would typically take between 15 and 20 hours on a 386 processor PC. For the direct search method, calibration time was about 10 % longer.

Computer time was reduced by looking over the model code and speeding it up, gradually sharpening the termination criteria for each parameter between loops and taking advantage of the model structure. Since the POC calibration order follows the direction of flow through the model, output from calibrated snow parameters could be stored. When calibrating the soil routine these data were read from a file instead of running the snow routine. A similar procedure was employed between the soil routine and the runoff-response function.

Computation speed could have been increased even further by reducing the number of parameters. This would involve reformulation of the model structure. Work along these lines is presently going on for the runoff-response function of the model.

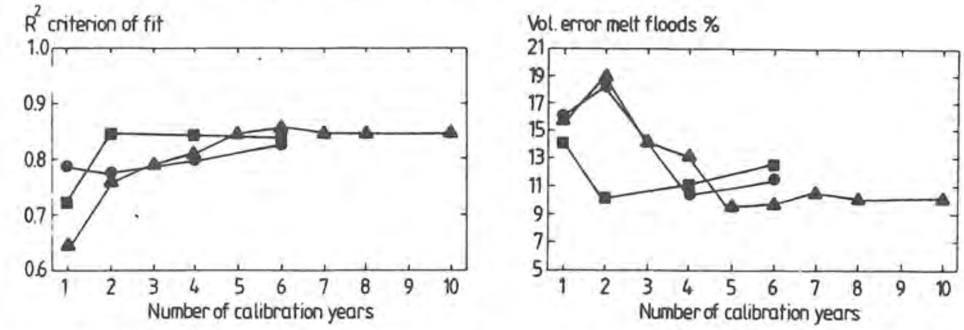

Fig. 10. Model performance after POC, at Torrön over a 20-year period for alternative lengths of the calibration period. The left figure shows the R^2 criterion of fit and the right figure shows relative volume error over snowmelt floods. Squares depict initially wet, circles initially average and triangles initially dry calibration periods.

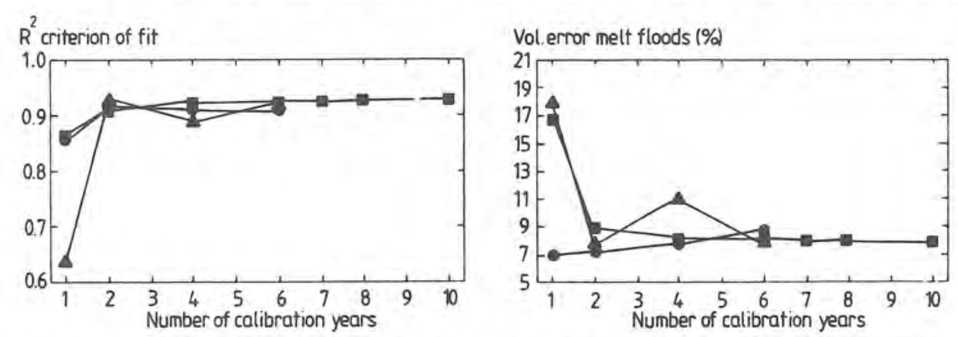

Fig. 11. Model performance after POC, at Trängslet over an 18-year period for alternative lengths of the calibration period. The left figure shows the R^2 criterion of fit and the right figure shows relative volume error over floods with a snowmelt contribution. Squares depict initially wet, circles initially average and triangles initially dry calibration periods.

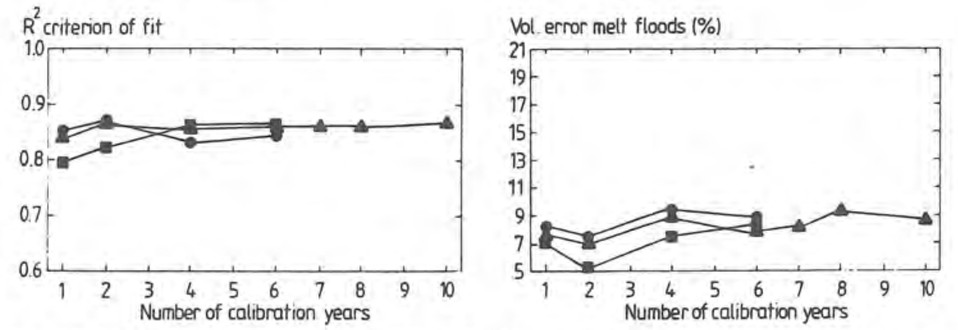

Fig. 12. Model performance after POC, at Simlången over a 20-year period for alternative lengths of the calibration period. The left figure shows the R^2 criterion of fit and the right figure shows relative volume error over snowmelt floods. Squares depict initially wet, circles initially average and triangles initially dry calibration periods.

Optimum Calibration Period Length

An advantage with automatic calibration is that different calibration periods can be tried systematically and objectively. As short calibration period as possible is desirable in order to reduce input data bases and to save computer time. But the shorter the period is, the larger is the risk of overfit, *i.e.* the verification period performance will be poor compared to the calibration period.

In order to find the optimum calibration period length the model was calibrated over alternative periods and verified over the total data period. The total data periods were 20 years for the Torrön and Simlången basins and 18 years for the Trängslet basin. The alternative calibration periods were formed by splitting the total period into three subsets of six years or more, so that one subset started with dry, one with average and one with wet years. To begin with, the first years of each subset were tried, then the following years within the subsets were added to form two-year periods and so on. One 7, 8 and 10-year period was also formed for each test basin.

Figs. 10, 11 and 12 show model performances at different calibration period lengths expressed in R^2 values and relative volume errors over the snowmelt floods. Triangles depict initially dry, circles initially average and squares depict initially wet subsets. After about four years all subsets represent average conditions. Optimum calibration period can be interpreted as the point at which the criterions level out.

Discussion and Conclusions

Calibration Scheme

The most straightforward way to compare calibration results is to compare the quality of the simulations visually. Since the inflow hydrograph contains a large amount of data of a range of types, *e.g.* rain floods, snowmelt floods, baseflow periods *etc.*, it is difficult to find one particular criterion that will objectively show the model performance. The R^2 criterion was chosen, since it shows how much of the initial variance the model explains. The accumulated relative volume error is interesting because it shows the error in water balance over the studied period. The volume error over the snowmelt floods is important when regulating hydropower reservoirs. In most Swedish rivers these floods constitute the majority of the yearly runoff.

The fact that POC gave slightly better model performance than manual calibration in terms of the criterions should not be overemphasized. As was illustrated in Figs. 7, 8 and 9, it is difficult to see the difference in quality between simulations giving different values. The conclusion is rather that POC yields a comparable model performance that is as good as after manual calibration.

Initially guessed parameter values in step one were far from the finally selected but gave fairly good results. The R^2 values from initially guessed parameters over the calibration periods varied between the basins and the length of the periods but were generally in the order of 0.5 to 0.7.

The POC scheme always converged to parameters with an overall good performance. In general three – four loops over the model were sufficient to find the optimal parameter set. In each loop the objective functions were evaluated about 120 times. This behaviour was consistent for all three test basins. The procedure coped well with the errors in inflow data. Observed inflow is normally computed as storage plus release. If the water level is misread or the reservoir oscillates, the storage and therefore also the inflow will be incorrect. These types of errors can clearly be seen in the inflow records for the Torrön reservoir, Fig. 7.

No significant changes in model performance were achieved by the finetuning experiments (less than $\pm$ 0.5 % on the R^2 criterion). From this it follows that the POC parameters were very close to an optimum. Unfortunately there are no direct methods of finding the global optimum, if one exist, and a check of the whole parameter space is unfeasible. For example if the twelve parameters that were calibrated could take on only ten values each, and if one evaluation of the objective function only takes one minute, a check of the whole parameter space would take approximately two million years.

The experiments with direct search starting from initially guessed parameters yielded surprisingly good results. The problem of conversion to unrealistic values was overcome by modifying the algorithm, so that the search always stayed within a realistic parameter space. As was shown in Table 3, performance criteria from direct search calibration were generally of the same magnitude as those from POC.

In the Simlången basin however, it was difficult to split the calibration period into subperiods dominated by one process. Perhaps this was the reason why the direct search method performed slightly better in this basin.

Optimal Calibration Period

The intention with trying different calibration periods was to check how efficient the calibration scheme was and to give a rule of thumb of how many years that normally would be needed. Figs. 10, 11 and 12 illustrate when an increase of the calibration period not further contributes to model performance. For the test basins the criterions levelled out at between two and six years. It was also seen that for certain one and two-year periods comparable results to those from ten-year periods were achieved. This indicated that even very short records of streamflow could be very useful in water resources planning if there are longer records of climate data. These records must, however, cover enough hydrological events.

Furthermore, testing different periods and several basins also emphasized the importance of the character of the calibration period and the fact that the amount of information for each parameter is different. The lower tank response function parameters (*PERC* and K_2) were active over long periods of time and very stable. Calibration of snowmelt parameters (*CFMAX* and *TT*), on the other hand, could only be made over a few events each year. This is illustrated by the irregular behaviour of the volume error of snowmelt floods for different calibration periods. Observe that no pure snowmelt floods could be isolated for the Simlången basin (Fig. 12). The volume error depicted for this basin is for floods with a snowmelt contribution.

Some model parameters required a couple of large floods to be correctly tuned, for example the upper tank threshold *UZL* and the highest recession coefficient K_0. At Simlången, floods occur throughout the year thus runoff from this basin contains more hydrological events each year to calibrate against. This is seen by the rapid levelling of the R^2 criterion for increased period lengths.

Parameter Values

In the introduction it was stated that the aim of calibration was to obtain a unique and conceptually realistic parameter set, so that the model becomes specific to the system it simulates. Manual, POC and direct search calibration converged to different parameter combinations, all conceptually realistic and with satisfactory output performance. One should therefore be careful in relating calibrated to measured parameter values. Interesting was that the following behaviour, with respect to parameter values, for all three test basins was observed;

1) Direct search often converged to parameter values close to the initial set. For periods shorter than four years this method sometimes failed to converge.
2) POC was not sensitive to initial parameter values and resulted in values similar to those from manual calibration.
3) In general the most instable parameters were those of the snow and soil routines, in particular the threshold temperature parameter *TT* and the soil parameter β. The upper tank threshold parameter *UZL* and the intermediate recession coefficient K_1, were the most instable parameters of the runoff-response function.
4) Large changes in parameter values were obtained for calibration periods between one and four years. For longer calibration periods the parameter values changed more gradually.

The POC scheme is straightforward, simple and consistently performed well. It was preferred to the direct search method, because it takes advantage of our understanding of the physical system, the model structure and the manual calibration experince. POC offers an automatic objective calibration method which should ensure more homogeneous results in, for example, design flood simulation.

There is always a risk of overemphasizing the aim of matching the model and the catchment response over the calibration period. Calibrating parameters only over time periods, where the physical processes they resemble are dominating, limits the degrees of freedom and reduces the risk of overfit. A model is never perfect, some errors should remain after a properly made calibration!

Acknowledgements

Ideas of how to automate calibraton of the HBV model have developed at SMHI for a long period of time. Almost after every manual calibration these issues are discussed anew. I wish to express my gratitude to my colleagues for sharing their experiences with me. I thank my supervisor, Professor Sten Bergström, who is responsible not only for developing the HBV model but also for many of the ideas behind this paper. Thank you also Göran Lindström, Magnus Persson and Kung Chen-Shan for your advice and constructive criticism.

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Parameter uncertainty and simulation of design floods in Sweden

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PARAMETER UNCERTAINTY AND SIMULATION OF DESIGN FLOODS IN SWEDEN

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Abstract

This paper examines the effects of parameter uncertainty on the simulation of recorded floods and design floods, using the HBV hydrological model. Two Swedish catchments with hydropower development are studied. A Monte Carlo procedure was used to generate parameter sets of different levels of uncertainty. The results showed that the most sensitive parameters in the calibration process were the snowfall correction factor and the recession parameters. Furthermore, when the model was extrapolated to simulate design flood and water stage hydrographs, the single most sensitive parameter was the highest recession coefficient. In addition, it was found that parameter uncertainty was associated with combinations of parameters rather than the absolute values of each.

Introduction

Design flood estimation is one of the most important factors for dam safety. Traditionally, design flood studies have been based on statistical analysis of observed flood records. Unfortunately, available records are usually much too short to enable frequency analysis of extreme design floods to be undertaken with any degree of confidence. In later years a new concept for design flood simulation, based on combining extreme climatological and hydrological factors, has gradually been adopted in many countries. The principle is that the probable maximum precipitation (PMP) over a catchment is combined with critical hydrological conditions to produce the probable maximum flood (PMF). This procedure is presently used for designing high risk dams in for example the USA, Great Britain and Norway (NRC, 1985; NERC, 1975; NVE, 1981 and 1986).

In Sweden, a similar procedure has recently been suggested (Swedish Committee on Spillway Design, 1990). The Swedish guidelines however, are not based on PMP estimates but on observed extreme areal rainfall in combination with extreme snowmelt simulations in a trial and error procedure until the worst flood for a specific system is found. The Swedish method of obtaining the design flood is to combine observed extreme hydrological factors to find a worst case scenario by simulation with a hydrological model, rather than to attempt through statistical analysis to obtain a flood magnitude corresponding to some arbitrarily chosen return period. Thus an unlikely extreme flood, but one well within the realm of possibility is generated.

In the iteration process, a 14-day design rainfall sequence, adjusted for season, catchment area and altitude, is successively entered into a hydrological model at alternative dates over a ten-year period. During the simulations the recorded rainfall over the catchment for the ten years is used, except for the 14-day period where the rainfall sequence is located. For the catchments studied in this paper, the unadjusted 14-day rainfall sequence was: 6, 6, 6, 6, 6, 10, 10, 40, 120, 25, 10, 10, 6 and 6 (mm), which gives a total of 267 (mm). Before the spring each year the soil is assumed to be brought to field capacity and the snowpack is replaced by an estimated 30-year snow value. All possible timings of the design precipitation over the ten year period are tried until the most critical spring and autumn floods are found. This means that some thousands of simulations have to be performed. The extreme flood hydrographs are then routed through the reservoir under study, employing a regulation strategy developed for each dam, to arrive at the design water stage hydrograph. For rivers developed for hydropower, with a system of dams, the design procedure becomes more complicated. Releases from upstream reservoirs have to be simulated and added to the local inflow to the reservoir being studied. The reservoirs studied in this paper however, are in the top reaches of their respective rivers, and all inflow to the reservoirs is unregulated.

The Swedish spillway design guidelines have been presented and discussed in a number of papers and at several international conferences for example by Bergström and Ohlsson (1988), Bergström, Lindström and Sanner (1989), Harlin (1989) and Bergström (1990a).

Although the Swedish design flood simulation procedure utilizes observed input conditions the particular combination of flood generating conditions has not been observed. From this it follows that the generated floods are larger than those observed and used to calibrate and evaluate the simulation model, i.e. an extrapolation is made. In Sweden the HBV hydrological model (Bergström 1976) is used. This paper addresses the question of the effect of model parameter uncertainty on resulting design flood and water stage hydrographs.

To test the sensitivity of the model output to the values of individual parameters the most appropriate method is to change the parameter under study by a few percent and evaluate the corresponding change in output. The effect of changing two parameters simultaneously can be shown by producing a response surface showing some expression of output difference as a function of the values of the two parameters. The expression of variation in the output may be taken as the sums of squares of difference from a standard output, or other definitions may be chosen. Calver (1988), Singh (1977), Arfi

(1980), Bergström and Forsman (1973) and Bergström, Brandt and Gustafson (1987), among others report on such sensitivity analysis. However, if the model parts are interdependent, the shape of the response surface from two studied parameters will depend upon the values assigned to the other parameters.

A common way to quantify the combined effect of parameter uncertainty on model performance is by first-order reliability analysis (e.g., see Garen and Burgess, 1981, and Lee, Georgakakos and Schnoor, 1990). This procedure is based on truncation, after the first two terms, of the Taylor series expansion of the model output as a function of its parameters. It is assumed that the mean model output is a function of the mean parameter vector. This is reasonable for approximately linear models. For highly nonlinear models, as most hydrological models are, the first order approximation can give misleading results. In such cases other methods have to be adopted. The most general is the Monte Carlo approach, in which repeated simulations are made using randomly generated parameter combinations. The Monte Carlo method is not limited by the complexity and non-linearity of the model. To obtain statistically reliable results a large number of runs must be made before convergence of the model output uncertainty is obtained, which is the major disadvantage of this approach. Sensitivity analysis based on Monte Carlo simulations have been performed by several authors, for example by Kung (1989), Gardner et al. (1980), Hornberger and Spear (1981) and Hornberger, Cosby and Galloway (1986).

The present work investigates the sensitivity of the HBV hydrological model outputs to different parameter combinations when the model is run in simulation and in extrapolation mode. Simulation mode is defined as running the model with true input. Extrapolation mode is when the model is run with critical combinations of extreme input, in order to generate design floods.

A Monte Carlo procedure similar to the methodology described by Hornberger et al. (1986) is used. Parameter ranges were compiled for each of two test catchments from parameter sets derived over different calibration periods and with different methods. The combined effect of uncertainty in parameter values only, was studied; the model structure itself was assumed to be correct.

The hydrological model and its parameters studied

The HBV hydrological model was developed by Bergström (1976). It is run on daily values of rainfall and air temperature and monthly estimates of potential evapotranspiration. The model contains routines for snow accumulation and melt, soil moisture accounting, runoff generation and a simple routing procedure. Each catchment is divided into zones according to altitude, lake area and vegetation. Bergström (1976) gives a detailed description of the model, only the parameters whose variations are studied and the corresponding model equations are given in this paper.

Snowmelt is calculated from a degree-day equation, according to:

$$Q_m(t) = CFMAX \cdot (T(t) - TT) \tag{1}$$

where: Q_m = snowmelt,
CFMAX= degree-day factor,
T = mean daily air temperature, and
TT = threshold temperature for snowmelt.

Because of the porosity of the snow, some rain and melt water can be retained in the pores. In the model, a retention capacity of 10% of the snowpack water equivalent is assumed. Only after that the retention capacity has filled, will melt water be released from the snow.

The rate of discharge of excess water from the soil is related to the weighted precipitation and the relationship depends upon the computed soil moisture storage, the soil saturation threshold (FC) and the empirical parameter β, as given in Equation 2. Small contributions of excess water from the soil result when the soil is dry and large contributions when conditions are wet.

$$Q_s(t) = \left(\frac{S_{sm}(t)}{FC}\right)^{\beta} \cdot P(t) \tag{2}$$

where: Q_s = excess water from soil,
S_{sm} = soil moisture storage,
FC = soil saturation threshold,
P = precipitation, and
β = empirical coefficient.

Evapotranspiration, is computed as a function of the potential evapotranspiration and the available soil moisture, as:

$$E_a(t) \begin{cases} = \dfrac{E_p \cdot S_{sm}(t)}{LP} & \text{if } S_{sm} \le LP \\ = E_p & \text{if } S_{sm} > LP \end{cases} \tag{3}$$

where: E_a = actual evapotranspiration,
E_p = potential evapotranspiration, and
LP = Ssm threshold for Ep.

Excess water from the soil and direct precipitation over open water bodies in the catchment area generate runoff according to Equations (4) and (5).

$$Q_u(t) \begin{cases} = S_{uz}(t) \cdot (K_0 + K_1) - K_0 \cdot UZL & if\ S_{uz} > UZL \\ = K_1 \cdot S_{uz}(t) & if\ S_{UZ} \leq UZL \end{cases} \tag{4}$$

$$Q_l(t) = K_2 \cdot S_{lz}(t) \tag{5}$$

where: Q_u = runoff generation from upper response tank,
K_0, K_1, K_2 = recession coefficients,
UZL = storage threshold between K_0 and K_1,
S_{uz} = storage in upper response tank,
PERC = percolation rate between the tanks,
Q_l = runoff generation from lower response tank, and
S_{lz} = storage in lower response tank.

The continuity equations over the separate model components are the following:

$$\frac{dS_{sm}}{dt} = P + Q_m - Q_s - E_a \tag{6}$$

$$\frac{dS_{uz}}{dt} = Q_s - Q_u - PERC \tag{7}$$

$$\frac{dS_{lz}}{dt} = PERC - Q_l + P_l - E_{pl} \tag{8}$$

where: P_l = precipitation over lakes, and
E_{pl} = potential evaporation from lakes,

Calibration parameters studied, included also the general snowfall correction factor (SFCF) which adjusts systematic errors in calculated snowfall and the transformation parameter MAXBAS, which is the number of days over which the generated flood pulse ($Q = Q_u + Q_l$) is distributed in time according to:

$$Q_{t+n} = \frac{4\ n + 2}{(MAXBAS)^2} \cdot (Q_u + Q_l) \tag{9}$$

where n = 0, 1, 2 ... ($\frac{\text{MAXBAS - 2}}{2}$)

When MAXBAS is an odd number, the weighting factor for the middle day is calculated as:

$$Q_{t+(MAXBAS-1)/2} = \frac{2 \cdot MAXBAS - 1}{(MAXBAS)^2} \cdot (Q_u + Q_l) \qquad (10)$$

Applications

Two Swedish catchments, developed for hydropower were studied, namely the Torrön catchment in river Indalsälven and the Trängslet catchment in river Österdalälven. These catchments represent two different hydrological regimes. Torrön is a mountainous catchment partly above the timber line. Runoff from this catchment is dominated by snowmelt floods in spring (April - June) but large rainfall floods also sometimes occur in autumn (September - December) and sometimes floods are experienced during winter (January - March). The mean annual inflow at Torrön is 48 (m^3/s) and the active storage capacity of the reservoir is 1180 (million m^3).

Trängslet on the other hand belongs to an inland regime and the catchment is covered by forest. At Trängslet the runoff also follows a clear seasonal pattern. Snowmelt floods dominate but occasionally also large rainfall floods are experienced. The Trängslet reservoir has an active storage capacity of 880 (million m^3) and the mean annual inflow is 64 (m^3/s). Figure 1 gives some key data and examples of the runoff patterns in the sites studied.

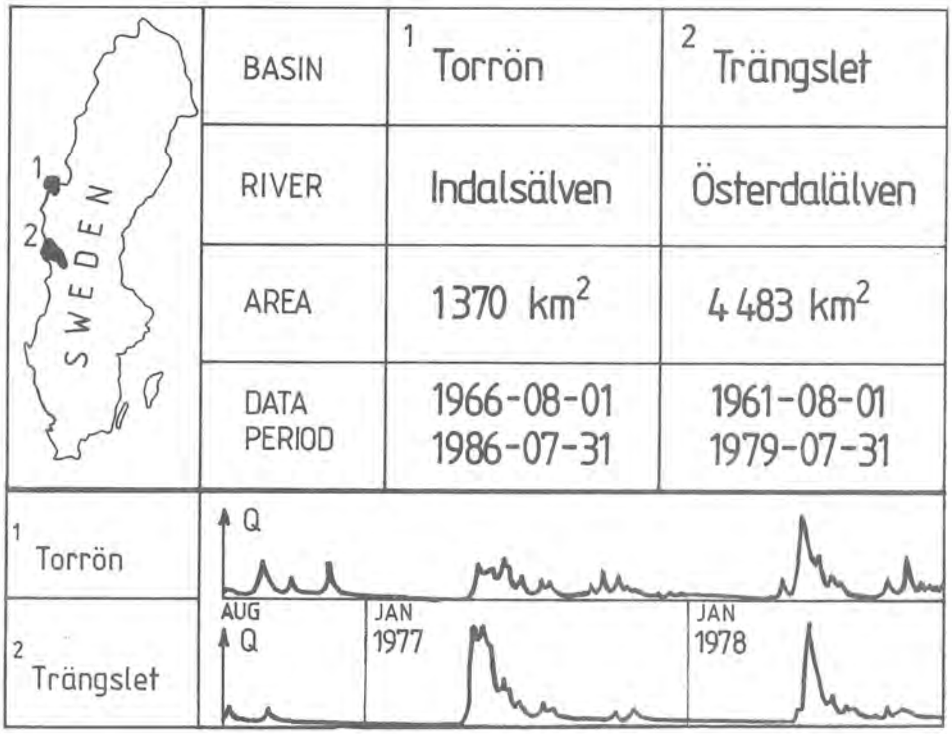

Figure 1. Key data and examples of the inflow pattern to the studied reservoirs.

Selection of parameter ranges

The HBV model is a conceptual model lumping many heterogenous catchment characteristics into simple linear and nonlinear equations. Although model components clearly represent individual hydrological processes, flow generating pulses should not be interpreted as emanating from exact locations in the catchment. The model formulation has been developed so that the integrated response of all flow pulses during a time step are captured. Parameter values are therefore integrated and specific for each catchment and can not be obtained from point measurements.

During the development of a process-oriented calibration scheme (POC) for the model, it was found that different calibration methods resulted in different optimal parameter settings (Harlin, 1991). Since no single parameter optimum exists, it was not considered meaningful to perform the sensitivity analysis close to one parameter set, as was done by for example Mein and Brown (1978). Instead, broad parameter ranges were formed by selecting the minimum and maximum values of each parameter from eight independent calibrations for each catchment (Table 1).

Table 1. Parameters and ranges used in the Monte Carlo simulation.

Parameter	Definition	Range (min / max)		Units
		Torrön	Trängslet	
Snow routine				
SFCF	Snowfall corr. factor	0.67 / 1.00	0.70 / 0.94	---
TT	Threshold temperature	-0.20 / +0.36	+0.14 / +0.80	°C
CFMAX	Degree-day factor	2.56 / 3.48	2.84 / 3.98	mm $°C^{-1}$ $24h^{-1}$
Soil routine				
FC	Maximum S_{sm}	50 / 274	132 / 274	mm
LP/FC	Ssm threshold for E_p	0.73 / 1.00	0.79 / 1.00	mm
β	Empirical coefficient	1.0 / 5.9	1.7 / 3.9	---
Upper response tank				
K_0	Recession coefficient	0.197 / 0.450	0.090 / 0.300	$24h^{-1}$
UZL	Storage threshold	12 / 44	15 / 63	mm
K_1	Recession coefficient	0.093 / 0.180	0.050 / 0.060	$24h^{-1}$
Lower response tank				
PERC	Filling rate	0.90 / 2.10	0.60 / 1.00	mm $24h^{-1}$
K_2	Recession coefficient	0.0008 / 0.05	0.0071 / 0.0250	$24h^{-1}$
Transformation function				
MAXBAS	Base length	1 / 2	2 / 3	24h

The parameter ranges were derived from manual calibration, direct search automatic calibration using Powell's conjugate gradient method (Powell, 1964) and from the POC scheme. A ten-year period was used for the Powell calibration. The POC scheme was used to produce six different parameter sets in each catchment; one set from calibrating over a ten-year period and five sets by calibrating over consecutive 4-year periods at Torrön and consecutive 3-year periods at Trängslet. According to Harlin (1991), the POC scheme can find acceptable parameter sets for such short periods in the Torrön and the Trängslet catchments. In this way a domain of acceptable parameter values was located.

Table 2 indicates the model performance over the complete data periods for the eight initial parameter sets expressed by the efficiency criterion (R^2, Eq. 11), (Nash and Sutcliffe, 1970), and the volume error (VE, Eq. 12) in percent.

Table 2. Model performance for the eight initial parameter sets over the total data periods; Torrön 20 yrs. and Trängslet 18 yrs.

Calibration method	TORRÖN			TRÄNGSLET		
	Calibr. period	R^2	VE	Calibr. period	R^2	VE
POC	66 - 70*	80.7	7.2	61 - 64	92.4	7.8
POC	70 - 74	79.8	10.7	64 - 67	92.2	5.5
POC	74 - 78	84.1	4.9	67 - 70	89.3	2.3
POC	78 - 82	83.0	7.7	70 - 73	89.0	1.8
POC	82 - 86	83.7	0.5	73 - 76	91.6	3.6
POC	76 - 86	84.8	1.7	61 - 71	92.8	4.1
Powell	76 - 86	84.0	5.0	61 - 71	90.5	2.9
Manual	all yrs.	78.8	2.7	all yrs.	92.0	0.8

*) = All periods start on 1/8 and end on 31/7.
POC = Process-oriented calibration (Harlin, 1991).
Powell = Conjugate gradient, direct search (Powell, 1964).

$$R^2 = 100 \cdot \left(1 - \frac{\sum_{i=1}^{n} \left(Q_m(i) - Q_o(i)\right)^2}{\sum_{i=1}^{n} \left(Q_o(i) - \overline{Q_o}\right)^2} \right) \tag{11}$$

$$VE = 100 \cdot \frac{\sum_{i=1}^{n} \left(Q_m(i) - Q_o(i)\right)}{\sum_{i=1}^{n} Q_o(i)} \tag{12}$$

where: n = total number of time steps,
Q_o = observed runoff,
$\overline{Q}_o$ = mean observed runoff,
Q_m = computed runoff.

Separate calibration and verification periods were not used.

The Monte Carlo procedure

For each parameter, 1000 values were randomly generated from a uniform distribution within the ranges of acceptable values. Parameter sets were formed by randomly combining the generated parameter values.

Simulation mode

For each catchment 1000 simulations were run over the complete data periods (20 years in the Torrön catchment and 18 years in the Trängslet catchment). For each run the R^2 criterion of model performance, volume error and maximum snowpack over the period were compiled. To permit statistical treatment of the results, it is important that the number of simulations be large enough. The output statistics; mean R^2 (R^2_m) and mean volume error (VE_m) were plotted after each realization (Figure 2). The convergence of R^2_m and VE_m shows that the number of realizations is enough.

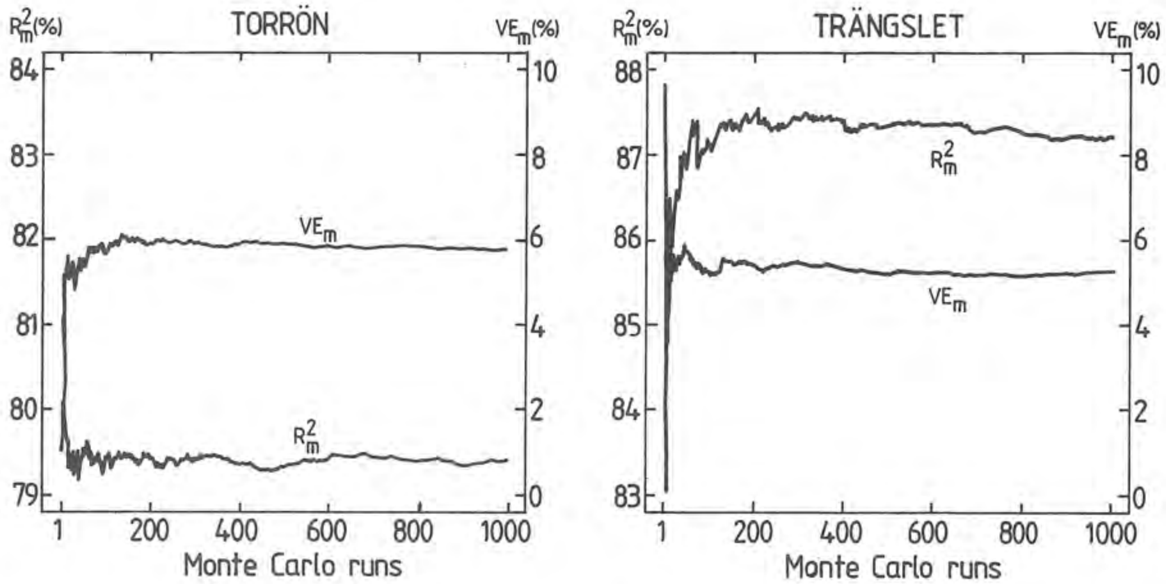

Figure 2. The mean R^2-value (R^2_m) and the mean volume error (VE_m) progression with the number of realizations.

Extrapolation mode

1000 simulations were also run in each catchment with critical combinations of extreme input to generate design floods. The critical timing of events was based on results from manual calibration. In doing so, the 14-day rainfall sequence was placed in time so that it generated:

1) the highest spring flood (first day of sequence: Torrön May 31, 1980, Trängslet May 18, 1978);
2) the highest spring flood water stage (first day of sequence: Torrön June 16, 1985, Trängslet May 22, 1978);
3) the highest autumn flood (first day of sequence: Torrön October 8, 1983, Trängslet August 8, 1985).

Since the prescribed reservoir operation strategy, for design flood calculations, is to fill the dam to the maximum pool level before August 1st, the highest autumn flood water stage normally results from the rainfall sequence timing giving the highest autumn flood peak. Which means that the rainfall sequence placing 3), was used to analyze both flood peak and maximum water stage for autumn conditions.

The thirty year snowpack, used in the design generation, is obtained from a frequency analysis of annual maxima snowpacks in the stated period, obtained by modelling in the simulation mode. As snowpack values are dependent on calibration, new 30-year snowpacks according to the current parameter set had to be derived before each Monte Carlo simulation. The 30-year snowpack was calculated from the maximum snowpack from the complete data period, simulated with the current parameter set, by a regression equation.

The regression equations between maximum snowpack and 30-year snowpack were found by running frequency analysis of annual maxima snowpacks for the eight initial parameter sets. Because the 30-year snowpack is of the same order of magnitude as the maximum snowpack over a twenty-year period, the effect of extrapolating the frequency function is small, and since different calibrations rescale all annual snowpacks in a similar manner, the relationship between the largest snowpack and the 30-year snowpack is almost linear. Equations 13 and 14 give the regression equations for the Torrön and the Trängslet catchments respectively.

$$S_{p30} = 66.9413 + 1.0601 * S_{pmax} \tag{13}$$

$$S_{p30} = 2.168 + 1.0083 * S_{pmax} \tag{14}$$

Where S_{p30} is the 30-year snowpack and S_{pmax} is the largest annual snowpack over the total data period.

For the Torrön catchment (Eq. 13), the explained variance of the regression is 99.6 (%) and for Trängslet (Eq. 14) 98.9 (%). In obtaining the regressions, the Type I Extreme Value (Gumbel) distribution was used with the maximum likelihood method of parameter estimation. All parameter combinations in the Monte Carlo simulation are within the parameter domain given by the initial parameter sets which implies that the regression equations above are not extrapolated beyond the observation points.

Results

Simulation mode

Figure 3 shows the relationship between the two model performance criteria (R^2 and VE), obtained for all Monte Carlo runs. Each simulation run was then classified as producing either acceptable or unacceptable results, according as the simulation produced at least as high R^2 value and as low volume error as the poorest of the initial parameter sets (Table 2). A large number of acceptable and unacceptable parameter sets were thus compiled. The key idea was to compare the distribution of individual parameter values between the acceptable and unacceptable results. If the two distributions were significantly different, the output is sensitive to the parameter being studied, i.e. it has a large effect on the model output, independent on how the remaining parameters are set. The shapes of distribution functions for the acceptable and unacceptable results also indicate if the chance of acceptable or unacceptable results is larger at any interval of the parameter range.

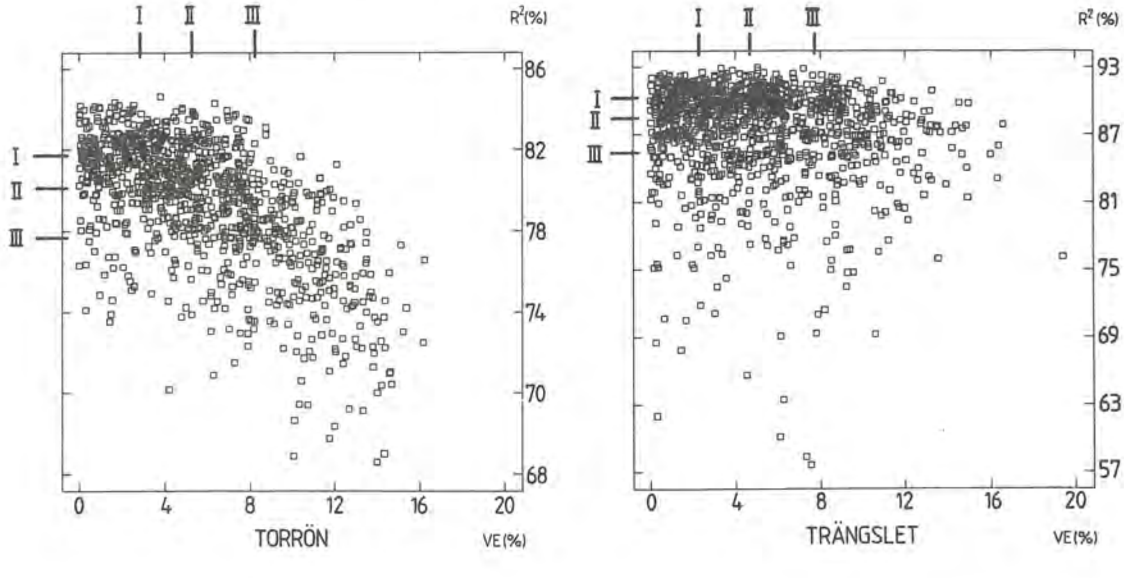

Figure 3. Model performance expressed with R^2 and volume error criteria of fit for the 1000 generated parameter sets. The evaluation period is 20 years at Torrön and 18 years at Trängslet. I, II and III define three levels of calibration accuracy used in studying the model performance during extrapolation mode.

The Kolmogorov-Smirnov two-sample statistic, $d_{m,n}$, was used to compare the cumulative distributions of the parameters which gave acceptable and unacceptable results. This statistic is normally used to check if two samples could have come from the same distribution. It calculates the maximum distance between the cumulative distribution functions and if this is large enough, the hypothesis that the two distributions are the same is rejected. This statistic has been used for ranking parameters by Hornberger et al. (1986), who concluded that when the Kolmogorov-Smirnov statistic is used for this purpose it should not: "be considered as a criterion for hypothesis testing in the traditional sense but rather as a useful index for ranking the importance of parameters in simulating the behaviors of interest".

Three sensitivity classes were selected to rank the parameters at hand; sensitive for $d_{m,n} \geq 0.2$, moderately sensitive for $0.1 \leq d_{m,n} < 0.2$, and insensitive for $d_{m,n} < 0.1$. Table 3 gives the sensitivity ranking of the parameters when running the model in simulation mode. Examples of the probability density functions for the three sensitivity classes are shown in Figure 4.

Table 3. Sensitivity of calibration parameters, based on the two-sample Kolmogorov-Smirnov statistic, $d_{m,n}$, when running the model in simulation mode. The parameters are classed as:
S = sensitive,
$d_{m,n} \geq 0.2$,
M = moderately sensitive,
$0.1 \leq d_{m,n} < 0.2$, and
I = insensitive $d_{m,n} < 0.1$.

Parameter	TORRÖN (n = 630, m = 370)	TRÄNGSLET (n = 359, m = 641)
SFCF	S	S
TT	I	I
CFMAX	S	I
FC	I	I
LP	I	I
β	I	M
K_0	M	S
K_1	M	S
UZL	I	M
K_2	I	M
PERC	I	I

n = number of acceptable simulation runs,
m = number of unacceptable simulation runs.

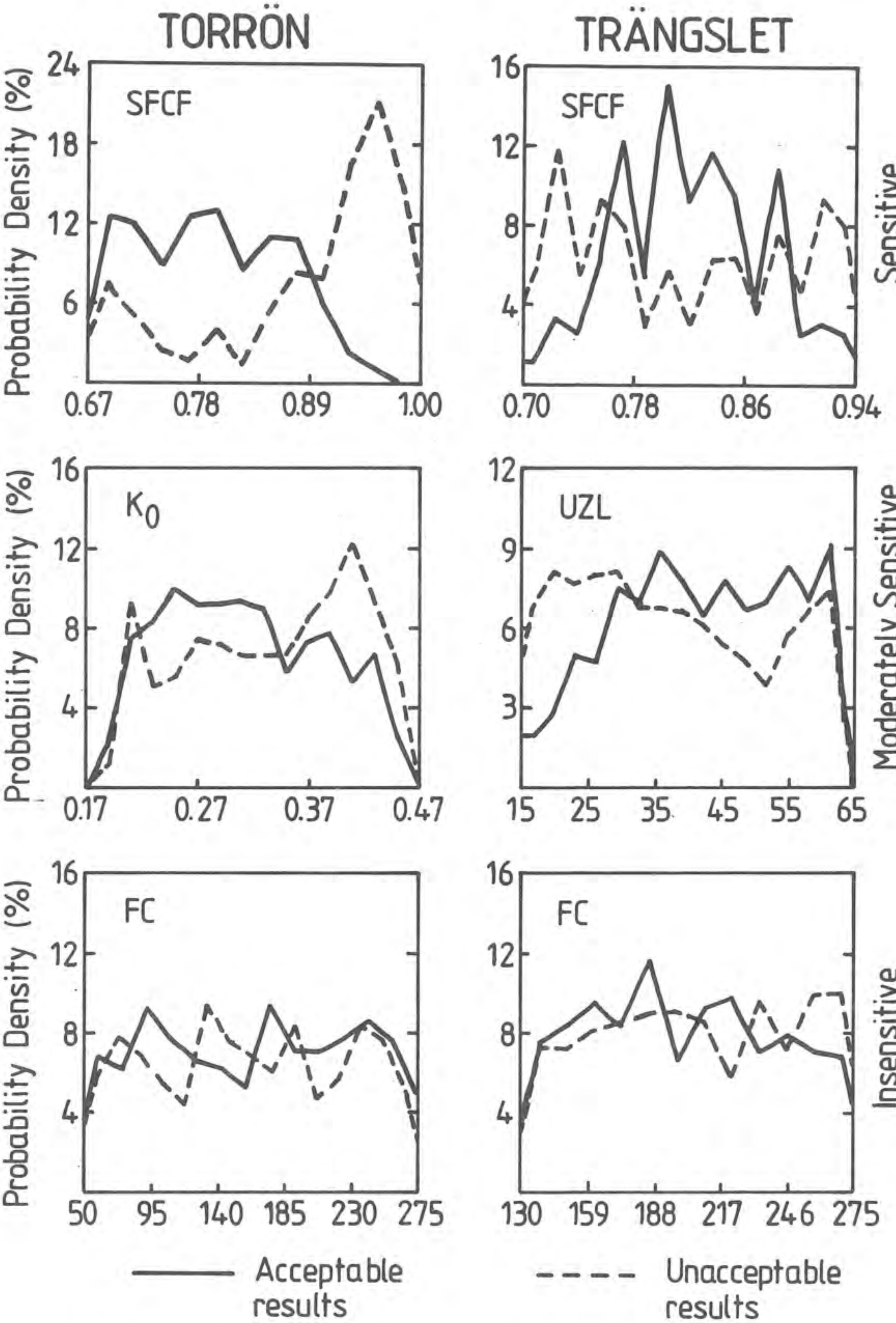

Figure 4. Examples of the parameter frequency distributions for acceptable and unacceptable results for the three sensitivity classes during simulation mode.

Extrapolation mode

The results from the analysis in simulation mode showed that model performance was associated with parameter combinations. Therefore it was necessary to group the parameter sets by levels of accuracy. This was done by combining the upper quartile, median and the lower quartile of the R^2 criterion of fit with the lower quartile, median and upper quartile of the volume error criterion, to form three performance levels (I, II, and III, Figure 3). Level I will thus represent a high level of parameter accuracy, II moderate parameter accuracy and III low accuracy, based on the two performance criteria R^2 and volume error.

Simplified regulation strategies were applied when routing the design floods through the reservoirs.

Before the onset of the spring flood, the Torrön reservoir was drawn down to the mean water stage for winters with plenty of snow (407.0 m.a.sl.). When the spring flood had raised the reservoir to 417.0 (m.a.sl.), release through the turbines up to a maximum of 165 (m^3/s) was assumed. Above 417.21 (m.a.sl.) the spillways were gradually opened, to be fully opened at the maximum pool elevation (417.46 m.a.sl.). This gave a release of 470 (m^3/s) at the maximum pool elevation, and if the reservoir continued to rise, the release at the top of the dam was 550 (m^3/s).

Before the arrival of the autumn flood, the Torrön reservoir was filled to the level 417.21 (m.a.sl.). When the reservoir started to rise, the same regulation strategy was applied as for the spring flood.

A slightly simpler regulation strategy was used for the Trängslet reservoir. The reservoir was lowered to the minimum pool elevation (388.01 m.a.sl.) before the onset of the spring flood. Between the minimum pool and the maximum pool elevation (422.95 m.a.sl.), 50 (m^3/s) were assumed released through the turbines. When the water level rose above the maximum pool elevation, the spillways were fully opened, thus releasing 875 (m^3/s) at the maximum pool elevation and 1 350 (m^3/s) at the top of the dam.

The autumn flood simulation at Trängslet started at the maximum pool elevation, and the regulation was done in the same manner as during spring.

For all simulations the reservoir storage curve and the spillway release curve were assumed valid also in the event that the water stage rose above the top of the dam during the simulation. Figures 5 and 6 show the sensitivity of maximum spring flood reservoir water stage, at Torrön and Trängslet, to parameter accuracy. The variance in maximum water level increases rapidly as the parameter accuracy decreases. To determine which parameters contributed most to the output variance in extrapolation mode, the Level I parameter sets were studied in more detail. Level I corresponds to an $R^2 \geq 81.8$ (%) with a $VE \leq 2.9$ (%) at Torrön and an $R^2 \geq 90.2$ (%) with a $VE \leq 2.3$ (%) at Trängslet.

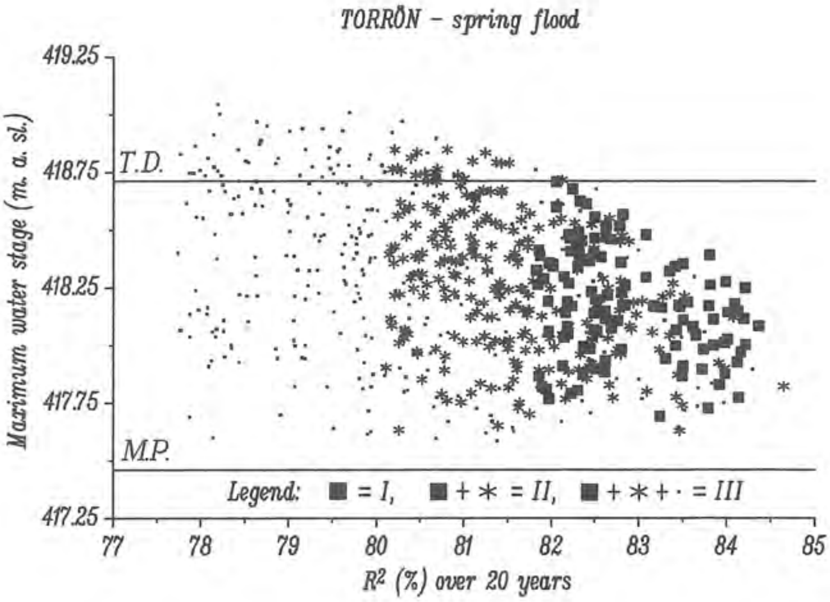

Figure 5. Maximum reservoir water stage for the most critical spring flood to Torrön at three levels of calibration accuracy (I, II and III). T.D. = Top of dam, and M.P. = Maximum pool elevation.

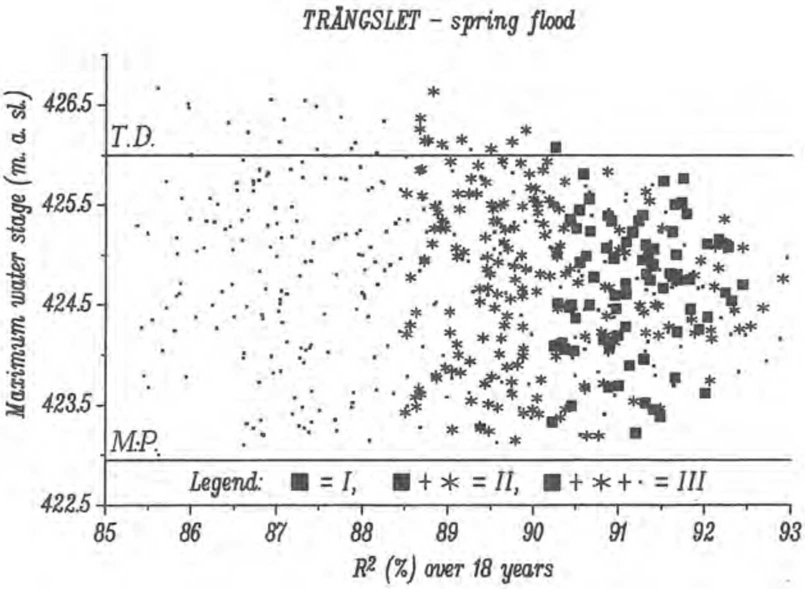

Figure 6. Maximum reservoir water stage for the most critical spring flood to Trängslet at three levels of calibration accuracy (I, II, and III). T.D. = Top of dam, and M.P. = Maximum pool elevation.

It was found that the upper recession coefficient K_0 was the single most sensitive parameter for both flood peak and water stage maximum in both catchments (Figures 7 and 8). The transformation parameter MAXBAS proved to be very sensitive in the Torrön catchment, which is clearly seen in Figure 7. To determine which parameters to pay special attention to when calibrating the model for design flood simulation, multiple regression was performed. The explained variance of the three first selected parameters in a stepwise forward selection procedure was calculated (Table 4).

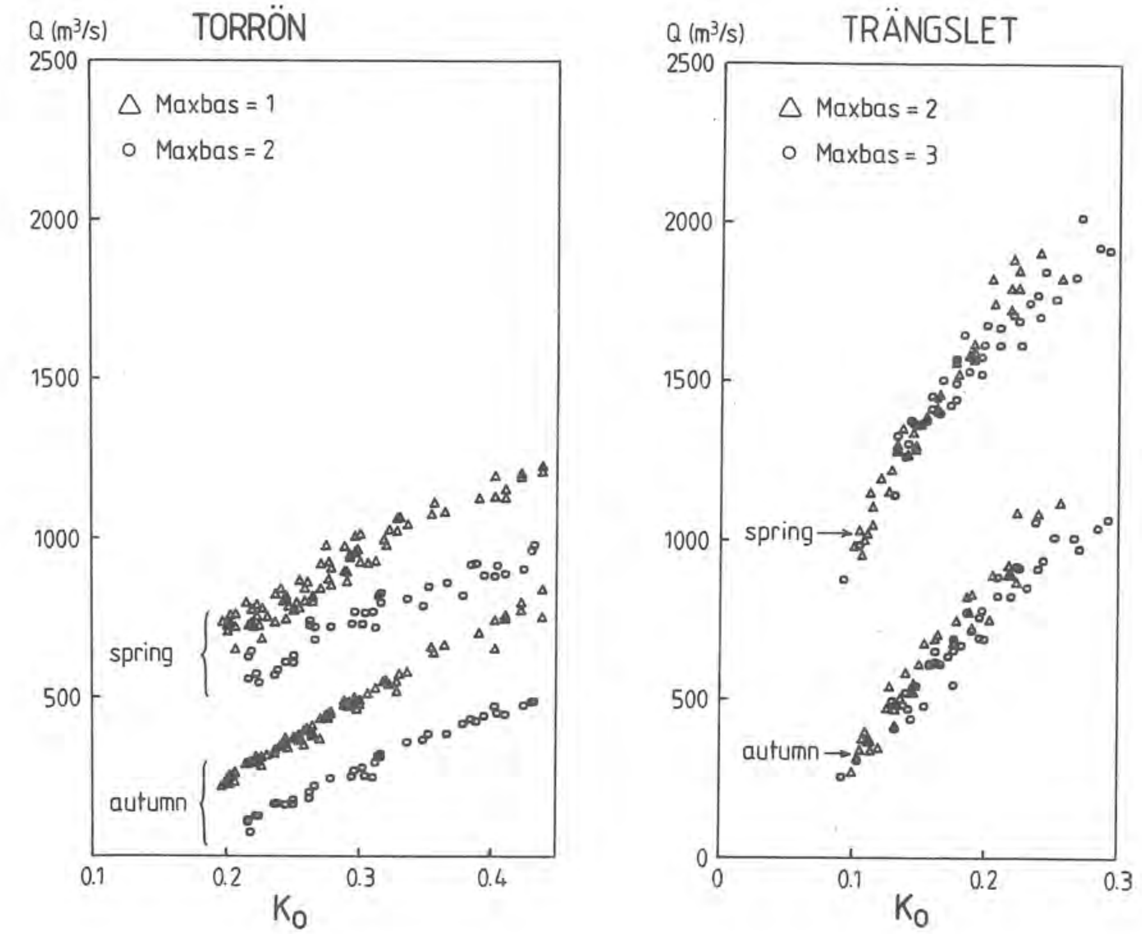

Figure 7. Design flood peak versus the highest recession coefficient K_0.

Flood volume, derived by summing the design flood hydrographs over 30 days, was analyzed and it was found that for the spring floods, the snowfall correction factor SFCF was the most important parameter.

It was also interesting to compare the initial parameters performance in extrapolation mode to the mean hydrograph from the Level I results. Figures 9 and 10 show the resulting hydrographs from manual calibration and POC calibration over a ten-year period together with the mean and 3-standard deviation bounds from the Level I results. Almost all of the Level I hydrographs were within the 3-standard deviation range.

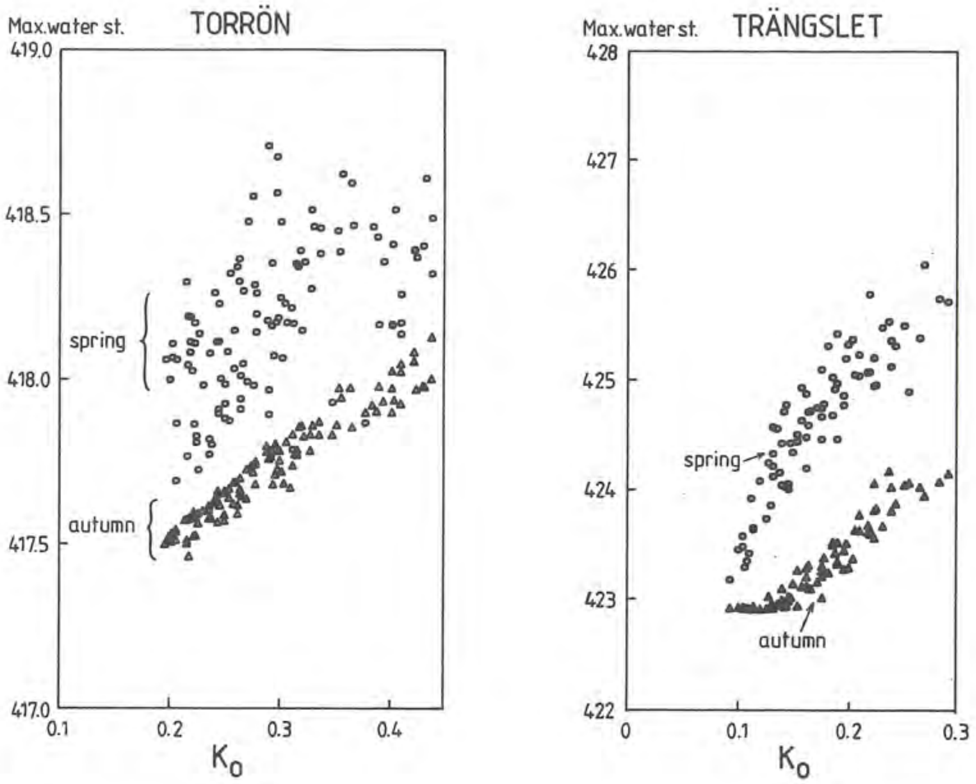

Figure 8. Design water stage maximum (m.a.sl.) versus the highest recession coefficient K_0.

Table 4. Explained variance (r^2) from stepwise forward selection multiple regression between maximum water stage (Wst_{max}) and flood peak (Q_{max}) against the model parameters.

	TORRÖN Parameters	TORRÖN r^2 (%)	TRÄNGSLET Parameters	TRÄNGSLET r^2 (%)
Q_{max}-spring	K_0 MAXBAS CFMAX	96.9	K_0 TT SFCF	95.2
Wst_{max}-spring	K_0 CFMAX SFCF	75.6	K_0 SFCF TT	90.1
Q_{max}-autumn	K_0 MAXBAS UZL	97.8	K_0 FC LP/FC	96.3
Wst_{max}-autumn	K_0 MAXBAS β	97.3	K_0 LP/FC FC	94.5

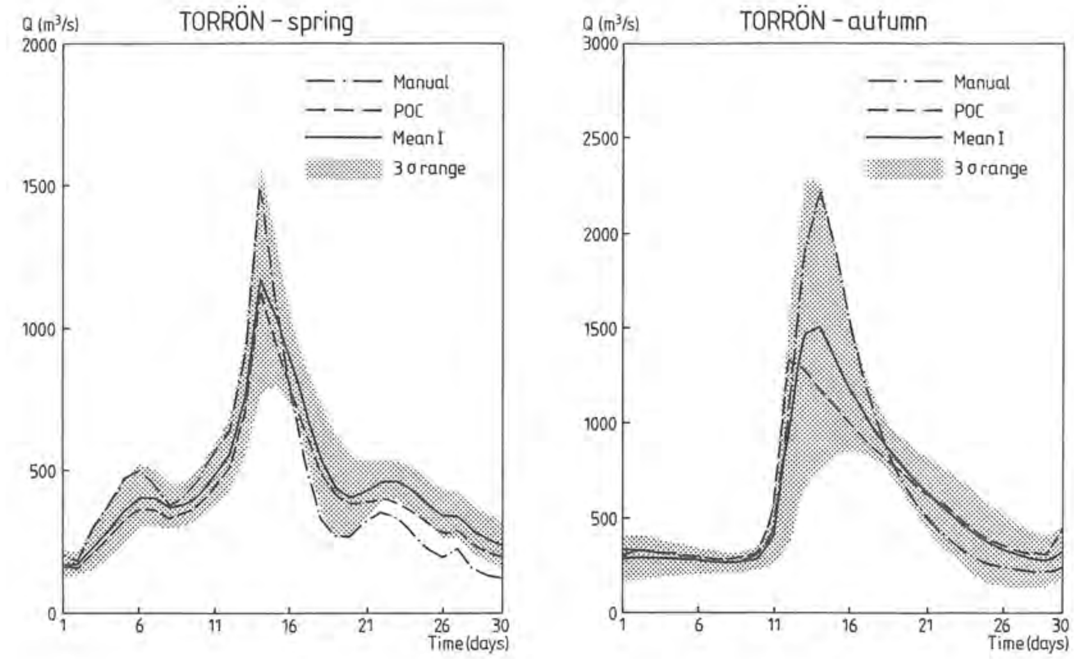

Figure 9. Design flood hydrographs for the Torrön catchment from manual calibration, POC calibration over 10 years and the mean of the Level I results. The shaded area shows the 3-standard deviation bounds for Level I.

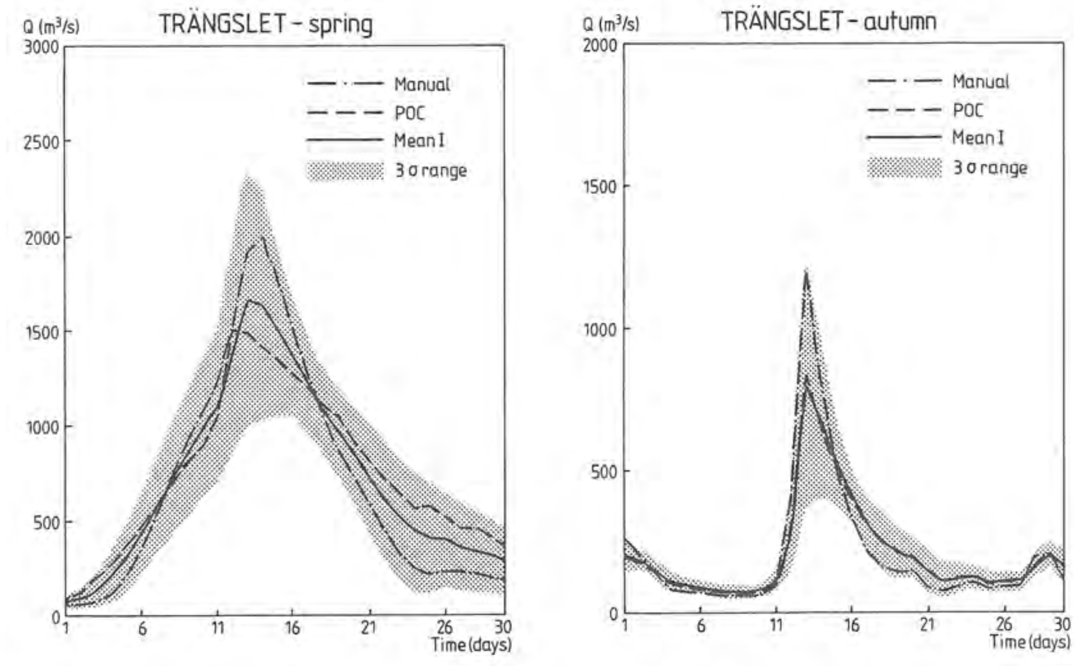

Figure 10. Design flood hydrographs for the Trängslet catchment from manual calibration, POC calibration over 10 years and the mean of the Level I results. The shaded area shows the 3-standard deviation bounds for Level I.

Discussion and conclusions

Simulation mode

The range of the parameters formed by calibrating with different methods and over different periods was large compared to the ranges from manual calibration compiled for the whole of Sweden (Bergström, 1990b). But good model performance was achieved with the parameters, and there are no physically based reasons for rejecting them. Not all parameters had large ranges however. The K_1 recession parameter at Trängslet and the PERC parameter for instance have fairly small ranges. A small range indicates that the parameter is well defined for the catchment. Our sensitivity analysis is limited to the parameter sensitivity within the ranges in Table 1.

The modelled runoff was most sensitive, in both catchments, to the snowfall correction factor, SFCF, Figure 4. This corresponds well with the manual calibration experience. The SFCF controls the snowpack and therefore the spring flood volume. For the Torrön catchment, modelled runoff was also sensitive to the degree-day factor, CFMAX. Since CFMAX determines the melting rate of the snowpack, this parameter will have larger importance in catchments with large snowpacks. The yearly snowpack in the Torrön catchment is on average about three times greater than that in Trängslet.

However, the rising limb of the spring flood is also to a large extent controlled by the recession parameters. To rank the relative importance between the recession parameters and the snow parameters by a traditional sensitivity analysis, where one parameter is changed at a time would be difficult. The advantage of the present analysis is that parameter combinations are generated across the full specified ranges. By comparing acceptable and unacceptable parameter distributions, the effects of parameter interdependence are included, and the sensitivity of each individual parameter is better described.

In the Trängslet catchment, the simulated runoff was sensitive to the highest recession coefficients, K_0 and K_1, whereas at Torrön the output was only moderately sensitive to these parameters, Table 3. This result also corresponds well with the manual calibration experience. For catchments with a runoff pattern dominated by few and large floods, like the Trängslet catchment, the effects of changes of these parameters are clearly seen.

For parameters with large interdependence, for example the soil parameters FC, LP and β, low sensitivity should be expected. The results showed that the probability of an acceptable/unacceptable result was almost uniformly distributed for these parameters within their ranges. Furthermore, it was seen that the distributions of acceptable versus unacceptable results overlap extensively, even for the most sensitive parameters. This indicates that for most parameters it was not the individual value of each parameter that determined if the simulation would be acceptable or unacceptable, but several combinations of the parameter values.

Parameter sensitivity was also related to the quality of the input data, the characteristics of the catchments studied and the performance criteria used. The Torrön catchment is smaller, quicker in response and has more snow and less evapotranspiration than the

Trängslet catchment. This is reflected in the differences in parameter ranges and sensitivity.

When visually inspecting the simulations it was found that at Torrön the combination of high R^2 and low VE was required for a good model performance, whereas at Trängslet the R^2 criteria alone reflected the model performance. The reason for this difference could be that at Trängslet, fewer and larger floods were experienced and the quality of the inflow data was higher.

The distinction of acceptable and unacceptable results was based on two rather crude statistical criteria. Optimally the analysis should have been based on visual inspection of all realizations. But even if this had been done, there is no clear limit in performance which would separate the acceptable and unacceptable. Different calibrations result in a gradual scale of model performance quality, so the separation criteria must always be to some extent subjective. A draw back with both the chosen criteria is that they show larger relative sensitivity to floods than to base flow periods. However, they are the most commonly used for model calibration in Sweden and were selected by WMO in an intercomparison of several runoff models (WMO 1986).

Extrapolation mode

Generating design floods with the HBV model requires special attention to the calibration. It was seen that the influence of the quickest recession parameter K_0 increased considerably when extrapolating beyond the range of the observed floods. To reduce the uncertainty, the calibration must emphasize the recessions of the largest floods. Preferably, large rain floods should be used, since the recessions then are undisturbed by melting snow. This is also clearly stated in the final report of the Swedish Committee on Spillway Design (1990), and is normally done when calibrating manually for a design flood simulation. The POC scheme, however, tries to find the parameter set that gives the smallest errors in average over the separate hydrograph components, with equal weight given to each flood. This procedure is not optimal for a design flood calibration, which is also illustrated by the hydrographs in Figures 9 and 10.

To overcome this difficulty the POC scheme could be modified so that the recession parameters are fine tuned over the largest floods only. But an automatic calibration should always be visually checked, since the R^2 and VE performance criteria are not sufficient criteria in a real design situation.

With the current structure of the HBV model, it is possible to calibrate the K_0 parameter only against the upper part of the flood hydrographs. The uncertainty in K_0 could be reduced, if this part of the model was reformulated so that the K_0-equivalent parameter were active over a larger portion of the flood hydrograph.

The results from the Torrön catchment show that the MAXBAS parameter has large influence on the flood peak, which in turn also affects the resulting water stage. Since the Torrön catchment has a shorter time of concentration than the Trängslet catchment, the sensitivity to MAXBAS is higher.

Another difference between the catchments when extrapolating design floods is the relative importance of the snow parameters. Using manually calibrated parameters, the 30-year snowpack over the entire Torrön catchment is 842 (mm), and the corresponding snowpack over the Trängslet catchment is 288 (mm). The correlation between snowpack, spring flood volume and water stage is high. Therefore, uncertainty in the calibration of the snow parameters has larger influence on the maximum spring flood water stage at Torrön than at Trängslet. This is also the major reason for the reduced linear relationship between K_0 and water stage depicted in Figure 8. One way to reduce the effect of calibration uncertainty on the 30-year snowpack would be to update the yearly snowpacks against observed runoff before the frequency analysis is performed. If this were done, the yearly snowpack would always give a correct spring flood volume. The modelled snowpack would thus be less dependent on calibration and the observed runoff would be utilized to a greater extent.

The water stage resulting from a given inflow hydrograph is obviously to a large extent controlled by the applied reservoir operation strategy. The reservoir operation strategies used in this paper are only examples of possible ways of operating the spillways. In the Swedish guidelines for spillway design it is prescribed that the release from the reservoirs should be started in an early stage of the flood development. The operation strategy has to be developed in close cooperation with the river regulation enterprise of the studied river. When such a strategy is used, the water stage will not rise as high above the maximum pool elevation as is shown in this paper.

In the analysis all the separate design flood simulations were made at fixed locations of the 14-day rainfall sequence. In a real design flood simulation, thousands of trials, with the sequence placed at different dates, over a 10-year period, have to be performed. This was obviously infeasible for all 1000 parameter sets. But, choosing the timing which gave the most extreme combinations on the basis of the manually calibrated parameters, will probably be valid for most of the parameter sets. In an ongoing study at SMHI, complete trial and error of all sequence locations, changing one parameter at a time, have been made. It was found that the most extreme combination, for different parameter sets, often resulted from the same rainfall sequence location.

Parameter uncertainty will have a large effect on resulting design floods and water stage hydrographs. But, the variance in the results can be removed by reducing the uncertainty in only a few parameters (Table 4), of which the highest recession coefficient, K_0, is the most important.

Acknowledgements

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VI

Modelling the hydrological response of extreme floods in Sweden

by

J. Harlin

Nordic Hydrology
(Submitted)

MODELLING THE HYDROLOGICAL RESPONSE OF EXTREME FLOODS IN SWEDEN

Joakim Harlin[1]

Hydrological models are today used for simulating extreme floods in the purpose of designing dams and spillways. In doing so, an extrapolation beyond the floods of the calibration period is made. This paper addresses this problem in connection to the HBV hydrological model. The model component describing flood dynamics, the runoff-response function, is studied. The methodology has been to calibrate different runoff-response functions over small to moderately large floods and to verify the performance over independent periods containing large experienced floods. Furthermore, the different model versions were run with extreme rainfall in order to generate design floods. It was found that the five parameter response function of the original HBV model could be replaced by nonlinear functions including fewer parameters. However, it was difficult to select any response function formulation as significantly better than the others when extreme floods larger than those of the calibration period were simulated.

INTRODUCTION

The hydrologic cycle consists of innumerable, extremely complex phenomena. Nature shows large variability, which makes generalization of small-scaled process studies to the catchment scale difficult. However, the dynamics of the rainfall-runoff processes can be approached from a system's viewpoint by considering the catchment as a hydrologic system. There are many definitions of a system. Chow et al. (1988) define a system as "*a set of connected parts that form a whole. The hydrologic cycle may be treated as a system whose components are precipitation, evaporation, runoff, and other phases of the hydrologic cycle.*" By using the system concept, investigation of input, output relationships are made rather than detailed process descriptions with exact physical laws. In many situations, the immediate need to solve a practical problem requires the use of a systems approach (Singh, 1988).

Hydrological models are approximations of the actual hydrologic system. The fundamental water balance equation for a catchment model is:

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$$Q(t) = P(t) - E_a(t) - \frac{dS}{dt} \tag{1}$$

where
- Q = discharge,
- P = precipitation,
- E_a = evapotranspiration,
- S = water stored within the catchment, and
- t = time.

This equation involves no approximation as long as no groundwater crosses the system boundaries, (Figure 1). The idea with hydrological modelling is to establish simple relationships between these variables, so that the water balance dynamics can be simulated. The most familiar and well established hydrological model in Sweden is the HBV model (Bergström, 1976).

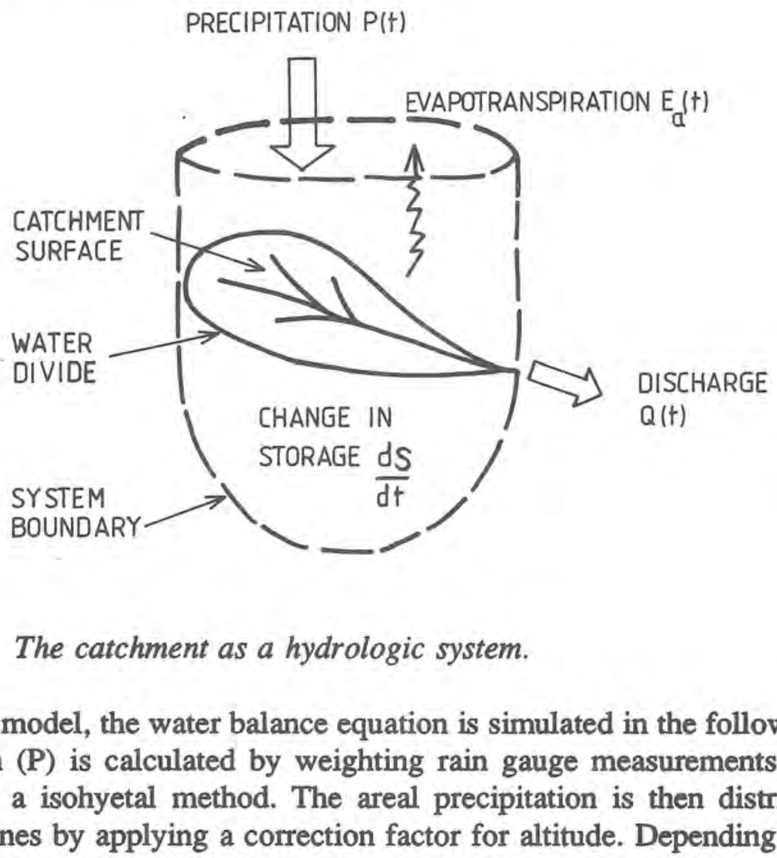


Figure 1. The catchment as a hydrologic system.

In the HBV model, the water balance equation is simulated in the following way: Areal precipitation (P) is calculated by weighting rain gauge measurements according to a Thiessen or a isohyetal method. The areal precipitation is then distributed over the elevation zones by applying a correction factor for altitude. Depending on whether the mean air temperature (T) is below or above a threshold temperature (TT), the precipitation will fall as snow or rain.

Snowpack (S_{sp}) is the first component constituting the general storage variable (S). When the temperature rises above a threshold value, the snowpack begins to melt. Meltwater (Q_m) is not released from the snow, however, until the retention capacity of the snow pores is reached. This capacity is normally set to 10 % of the snowpack water equivalent. Snowmelt is calculated with a degree-day equation, according to:

$$Q_m(t) = CFMAX \cdot (T(t) - TT) \tag{2}$$

where CFMAX = the degree-day factor.

The general storage variable (S) is also formed by the soil moisture storage (S_{sm}). Percolation of excess water from the soil moisture storage (Q_s) is related to the precipitation and the computed soil moisture storage as given in Equation 3. Rain or snowmelt generate small contributions of excess water from the soil when the soil is dry and large contributions when conditions are wet.

$$Q_s(t) = \left[\frac{S_{sm}(t)}{Fc}\right]^{\beta} \cdot P(t) \tag{3}$$

where Fc = soil saturation threshold and
β = model parameter.

Evapotranspiration (E_a) is computed as a function of the soil moisture conditions and the potential evapotranspiration (E_p). However, when the soil moisture exceeds a storage threshold (Lp), water will evaporate at the potential rate. The equations are:

$$E_a(t) \begin{cases} = \dfrac{E_p \cdot S_{sm}(t)}{Lp} & if\ S_{sm} \leq Lp \\ = E_p & if\ S_{sm} > Lp \end{cases} \tag{4}$$

Snow and soil moisture modelling is made separately for each type of land use and elevation zone. But the runoff generation is formed by transforming excess water from the soil plus direct precipitation over open water bodies with a lumped runoff-response function. It results from the assumption that the catchment response behaves like two linked tanks (Figure 2). The lower tank is a linear reservoir, representing base flow. It is filled by percolated water from the upper tank (PERC) plus precipitation over open water bodies (P_l) and responds with discharge and evaporation from lakes (E_l).

If the excess water from the soil exceeds the percolation capacity, the upper tank starts to fill. This tank simulates the catchment response to flood events and has two recession coefficients separated by a threshold in storage (Figure 2). Storage in the response function is the third component of the general storage variable (S) in Equation (1). Thus the total storage $S = S_{sp} + S_{sm} + S_{uz} + S_{lz}$.

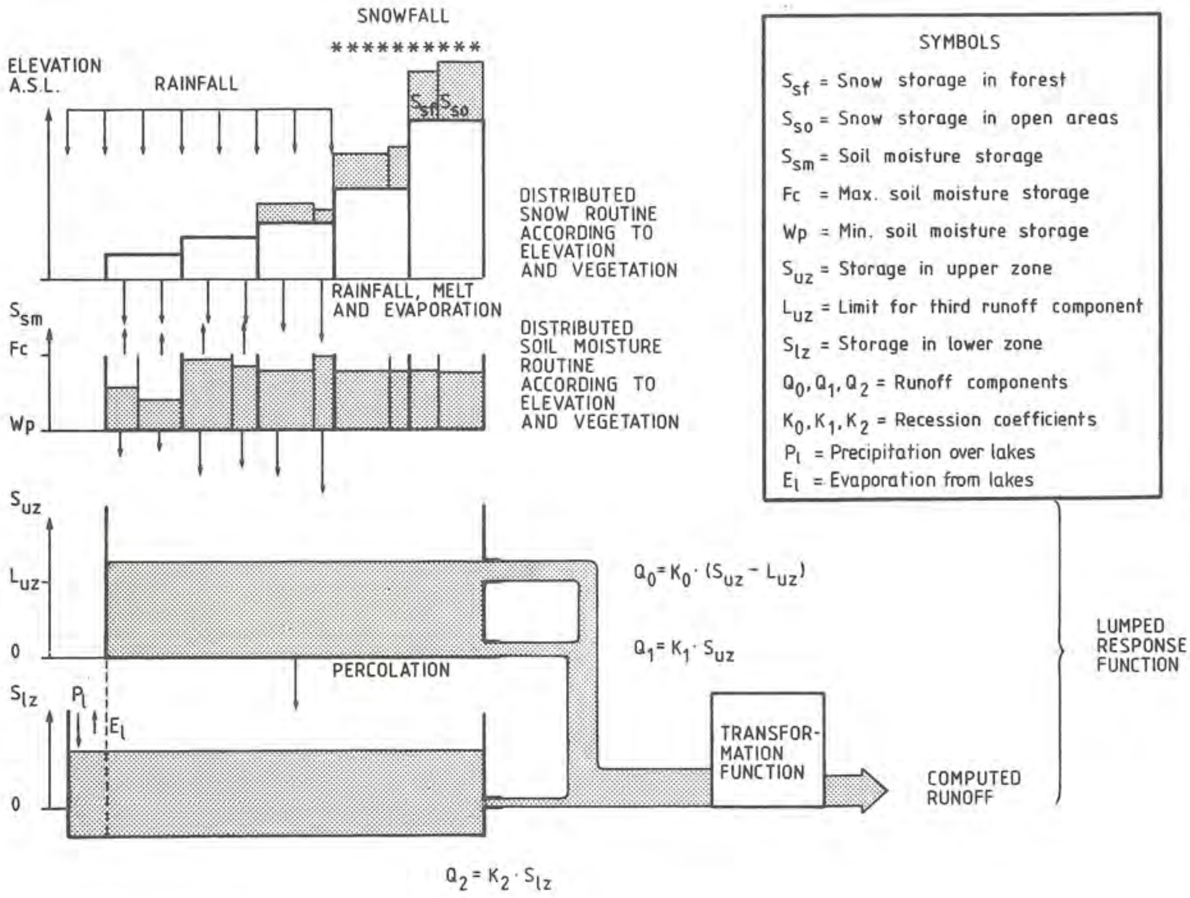


Figure 2. *The basic structure of the original HBV model.*

By simulating the hydrological cycle conceptually as described, one is forced to calibrate the model parameters by matching the model output with the observations. In order to get stable and representative parameter values it is essential to restrict the parameter number and to formulate the model equations so that the parameters are independent of each other. In model development and calibration it is important to split the available data set and save an independent period to verify the model performance. If this principle is followed and repeated for several areas, confidence in the model formulation can be gained (Bergström, 1991).

Hydrological models are today not only used for regular inflow forecasting to reservoirs, but also for extending runoff series, filling in gaps of missing data, studying the effects of a hypothetical change in climate, and for simulating extreme floods for designing dams and spillways. This means that the modeller takes a step into the unknown and the results can not be checked against observations. In Sweden, the HBV model is currently used for simulation of design floods. These floods are well beyond the observed floods used in the calibration process (Bergström et al., 1989). Consequently, an extrapolation is made and model structure and calibration will influence the magnitude of the generated design floods.

The objective of this paper is to study catchment rainfall-runoff response and develop simple runoff-response functions with few calibration parameters. The problem of modelling extreme floods, well beyond the range of those of the calibration period is focused. The methodology has been to calibrate different runoff-response functions over small to moderately large floods and to verify the performance over independent periods containing large experienced floods. Furthermore, the model versions were run with extreme rainfall in order to generate design floods.

STUDY CATCHMENTS

Six catchments were studied, namely: Torrön, Äcklingen, Alfta, Ljusnedal, Trängslet and Torsebro (Figure 3).

CATCHMENT	(km2) AREA	RIVER
TORRÖN 1)	1370	INDALSÄLVEN
ÄCKLINGEN 2)	157	RUTSÄLVEN
LJUSNEDAL 3)	340	LJUSNAN
TRÄNGSLET 4)	4483	ÖSTERDALÄLVEN
ALFTA 5)	3140	VOXNAN
TORSEBRO 6)	3676	HELGEÅN

Figure 3. Location and key data for the studied catchments.

These catchments represent different parts of Sweden and their runoff records contain some exceptionally large rain floods. The Äcklingen and Ljusnedal catchments are fairly small and quick in response. For these catchments, recession analysis was performed and experiments with a large number of response function formulations were carried out (Table 1). In the hydropower developed catchments; Torrön, Trängslet and Torsebro the HBV model is operationally used for inflow forecasting to the reservoirs. At Alfta the model is used for flood forecasting and flood warning. Two of the response functions developed from the behavior of Äcklingen and Ljusnedal were calibrated and run over extreme rain floods at Torrön, Alfta, Trängslet and Torsebro.

RECESSION ANALYSIS

The shape of the hydrograph is a function of the catchment characteristics and the meteorologic/hydrologic inputs and outputs over time. It is normally analyzed as consisting of a base flow component and a flood component. The flood component is the portion of the hydrograph that responds quickly and is clearly related to a given storm or snowmelt period. Base flow is formed by the continuous outflow of ground-water and lake discharge in the catchment.

The receding limb of the hydrograph is strongly related to storage and change in storage in the catchment after that the rainfall or snowmelt stops. The recession curve is often expressed as:

$$Q_t = Q_0 \, e^{-K \cdot t} \tag{5}$$

where Q_t = the discharge at time t,
Q_0 = the discharge when t = 0, and
K = the recession coefficient.

Taking logarithms, equation (5) becomes:

$$\ln Q_t = \ln Q_0 - Kt \tag{6}$$

which can be plotted as a straight line with the gradient -K in semi-logarithmic scale. Equation (5) is derived from a linear storage to outflow relation, equivalent to the formulation for the lower tank of the HBV model. On this basis many studies to define and analyze recession constants have been made (e.g. Bako and Hunt, 1988; Tallaksen, 1989; Schwarze et al., 1989; Petras, 1986 and Brandesten, 1987). Nathan and McMahon (1990) presents an evaluation of several automated techniques for base flow separation and recession analyses. A linear storage to outflow relation is normally not appropriate for modelling flood periods because K is rarely constant throughout a flood recession. In order to simulate the increase in recession rate with increasing storage, the HBV model contains an upper tank with two outlets.

A model with storage thresholds will be sensitive to the magnitude of the observed floods used for calibration. The highest recession coefficient (K_0) in the HBV model, for example, is only active on the upper part of the flood hydrograph. But, when extreme floods are simulated, this parameter will dominate the model response. Since the HBV model response function has five calibration parameters and includes a storage threshold (L_{uz}), the calibration can result in several parameter combinations with equally good performance over a calibration period, but with considerable differences when simulating extreme floods (Harlin and Kung, 1992). Gan and Burges (1990) arrived at similar conclusions. They simulated hydrologic response from extreme rainfall with a modified version of the Sacramento model. Their analysis was based on simulations for small (0.1 - 0.2 km^2) hypothetical homogeneous catchments. As reference, the output from the S-H model (Smith and Hebbert, 1983) was used. They stressed the problems with storage thresholds and the dangers of simulating floods beyond the calibration range.

Figure 4 shows recessions for undisturbed rain-generated floods, plotted as $\log_e$ discharge versus time for the Äcklingen and Ljusnedal catchments. It is evident from the recession plots that the slope of the curves change with discharge magnitude, hence a single linear relationship between storage and outflow for flood periods does not hold true. Lundin (1982) and Lind and Lundin (1990) report that for Swedish tills the saturated hydraulic conductivity and groundwater flow are high close to the soil surface and decrease rapidly in deeper layers. At high storage the subsurface flow will occur close to the surface, hence an increase in recession rate should be expected.

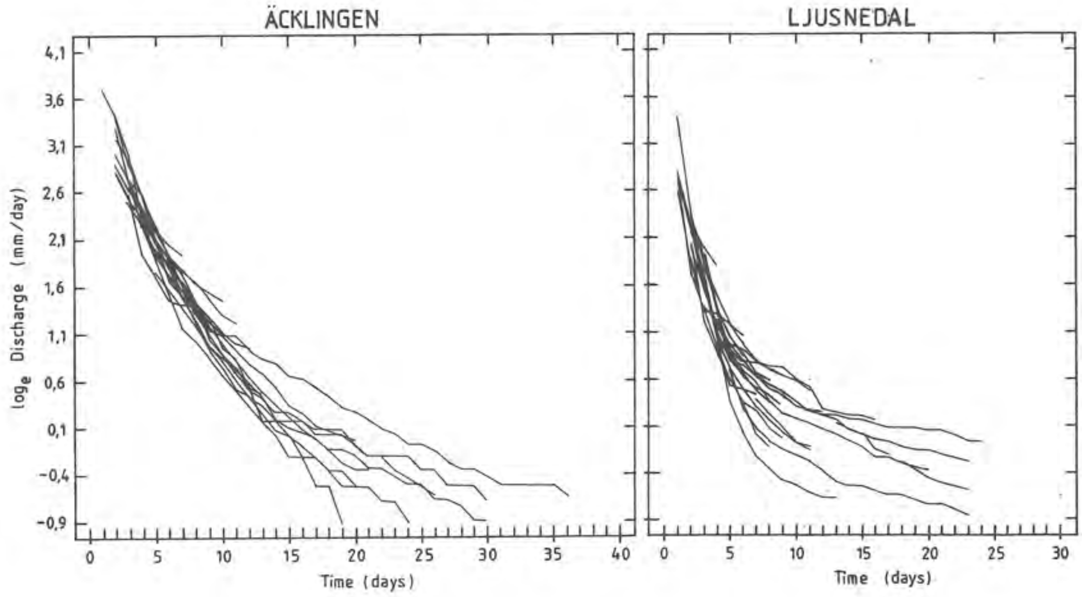


Figure 4. Recession curves for rain floods as ln (discharge) versus time plots.

If we draw an analogy with the hydraulics of channel flow, it is also seen that the flow convergence increases with increasing storage. Considering the Manning equation for a broad channel, the flow per meter channel width is:

$$q = \frac{1}{n} \cdot S_f^{\frac{1}{2}} \cdot y^{\frac{5}{3}} \tag{7}$$

where n = Manning roughness coefficient,
S_f = friction slope, and
y = water depth.

The equation for runoff from a unit width surface of the length, L, will be

$$Q = \frac{S_f^{\frac{1}{2}}}{n \cdot L} \cdot y^{\frac{5}{3}} \tag{8}$$

which is the equation of a non-linear reservoir,

$$Q(S) = K \cdot S^{\alpha} \tag{9}$$

with $K = S_f^{1/2}/(n \cdot L)$ and the power $\alpha = 5/3$. Equation (9) is also equivalent with the function for a single-valued rating curve used for discharge measurement.

SELECTION OF ALTERNATIVE RUNOFF-RESPONSE FUNCTIONS

Eight alternative runoff-response equations with non-linear storage to outflow properties, including fewer free parameters compared with the original HBV runoff-response function were formulated. Comparisons with the original HBV model equations by simulations in the catchments Äcklingen and Ljusnedal were made. The alternative runoff-response functions were both two tank and single tank functions, Table 1. Precipitation and evaporation from open water bodies was included in all alternative functions so that the remaining HBV model structure could be kept intact.

The test runs at Äcklingen and Ljusnedal showed that most of the alternative functions could be calibrated to simulate flood periods with comparable results to the original HBV model.

However, Function 1, which has an identical lower tank as the HBV model but an upper tank described by a single non-linear reservoir was clearly less capable of modelling flood periods, compared with the original HBV model. Furthermore, the single tank Functions 2, 3 and 4 gave poor model agreement over base flow periods and were therefore less general in application.

Function 5 though, was easily calibrated and resulted in good model performance both for base flow and flood periods. This function was selected for further analysis and will in the following be referred to as the "E-box".

Function 6 was suggested by Lindström et al. (1990). This equation will respond with a linear recession rate at low storage values and an increasing recession rate with increasing storage. However, the recession rate increases tremendously fast with storage which makes the equation difficult to calibrate so as to model both base flow and flood periods correctly. To moderate the increase in recession this equation was slightly modified, runoff-response function 7. Function 7 was selected for further analysis and will in the following be referred to as the "Ln-box".

Table 1. The alternative runoff-response functions, to the HBV runoff-response function, tested at Äcklingen and Ljusnedal.

Runoff-response function	Number of tanks	Equations	Calibration parameters
1	two	*Upper tank:* $Q_u = K \cdot S_u^{\alpha}$ *Percolation between the tanks* = *PERC* *Lower tank:* $Q_l = K \cdot S_l$	K, α PERC
2	one	$Q = K \cdot T + K \cdot (S - T)^{\alpha}$ *if* $S > T$ $Q = K \cdot S$ *if* $S \leq T$	K, T α
3	one	$Q = e^{\alpha \cdot S} - 1$	α
4	one	$Q = e^{\alpha \cdot S^{\beta}} - 1$	α, β
5	two	*Upper tank:* $Q_u = e^{K_1 S_u} - 1$ *Percolation between tanks* = *PERC* *Lower tank:* $Q_l = K_2 \cdot S_l$	K_1, K_2 PERC
6	one	$Q = K \cdot S^{(1 + \alpha \cdot S)}$	K, α
7	one	$Q = K \cdot S^{(1 + \alpha \cdot \ln(S))}$ *if* $S > 0$ $Q = 0$ *if* $S = 0$	K, α
8	one	$Q = K_1 \cdot S + K_2 \cdot S^2$	K_1, K_2

Function 8 is a polynomial expression. It resulted in good model performance but was difficult to calibrate since the parameters have a large interdependence.

Table 2, shows the model performance for the HBV model with its original runoff-response function and the two alternative functions expressed by the R^2 criterion of fit, Equation 10, (Nash and Sutcliffe, 1970). All three functions were automatically calibrated over all flood and base flow periods with the POC calibration scheme (Harlin, 1991).

Table 2. Model performance, expressed as R^2 values (%), for the original HBV model and the selected alternative runoff-response functions.

	Äcklingen		Ljusnedal	
	Cal.	Ver.	Cal.	Ver.
HBV	87.3	86.7	80.7	86.4
E-box	87.0	86.2	79.4	83.6
Ln-box	86.6	86.0	78.5	79.9

Cal. = calibration period (1971-08-01 -- 1981-07-31)
Ver. = verification period (1981-08-01 -- 1989-07-31)

$$R^2 = 100 \cdot \left(1 - \frac{\sum_{t=1}^{n} \left(Q_m(t) - Q_o(t)\right)^2}{\sum_{t=1}^{n} \left(Q_o(t) - Q_{om}\right)^2} \right) \tag{10}$$

where n = number of time steps,
Q_m = modelled runoff,
Q_o = observed runoff, and
Q_{om} = mean observed runoff.

RESULTS

Simulation of observed extreme floods

The E-box and the Ln-box will have a non-linear increase in discharge with increasing storage, Figure 5. This property will result in an increasing recession rate with flood magnitude when modelling extreme floods, Figure 6.

In order to check the uncertainty due to model structure when modelling extreme floods, the three runoff-response functions: HBV, E-box and Ln-box, were calibrated on small to moderate rain floods and verified on large observed rain floods at Torrön, Alfta, Trängslet and Torsebro, Figures 7 and 8. Only rain floods were studied in order to reduce the influence of the snow routine of the model, and thereby receive a clearer picture of the runoff-response function behavior. During the simulations the original HBV model structure was used and only the runoff-response function was replaced when testing the E-box and Ln-box.

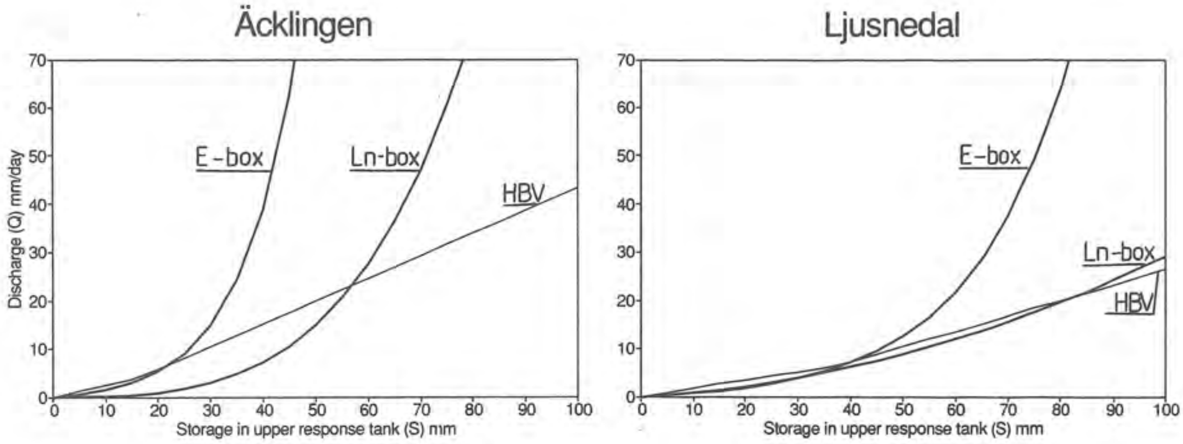


Figure 5. ***The discharge to storage relation for the studied runoff-response functions. Only the storage in the upper tank is depicted for the HBV and E-box. For the Ln-box (which is a single tank formulation) the total storage is shown.***

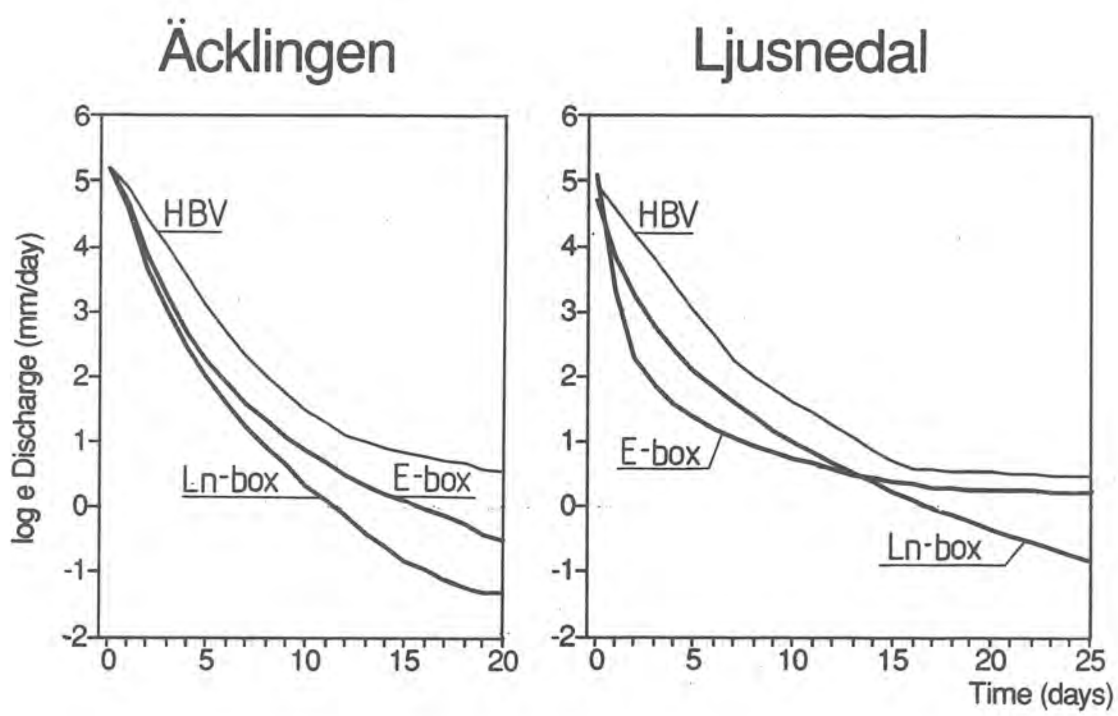
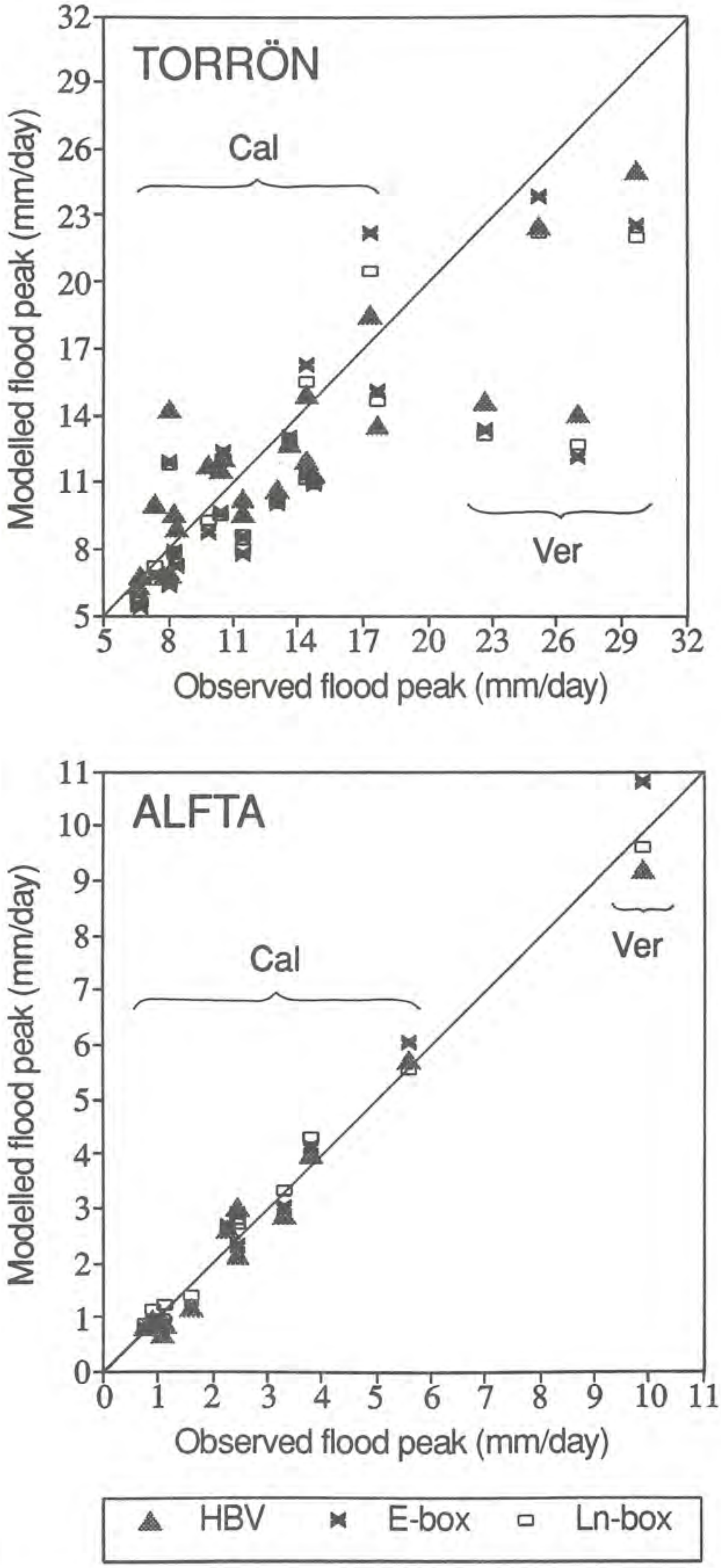


Figure 6. ***Recession curves for the three studied functions as ln (discharge) versus time plots.***



R^2-values
(10 years)
HBV 0.84
E-box 0.83
Ln-box 0.84

R^2-values
(8 years)
HBV 0.82
E-box 0.84
Ln-box 0.83

Figure 7. Model performance for the tested runoff-response functions at Torrön and Alfta. The functions were calibrated and verified on rain floods only. R^2 values refer to calibration periods.

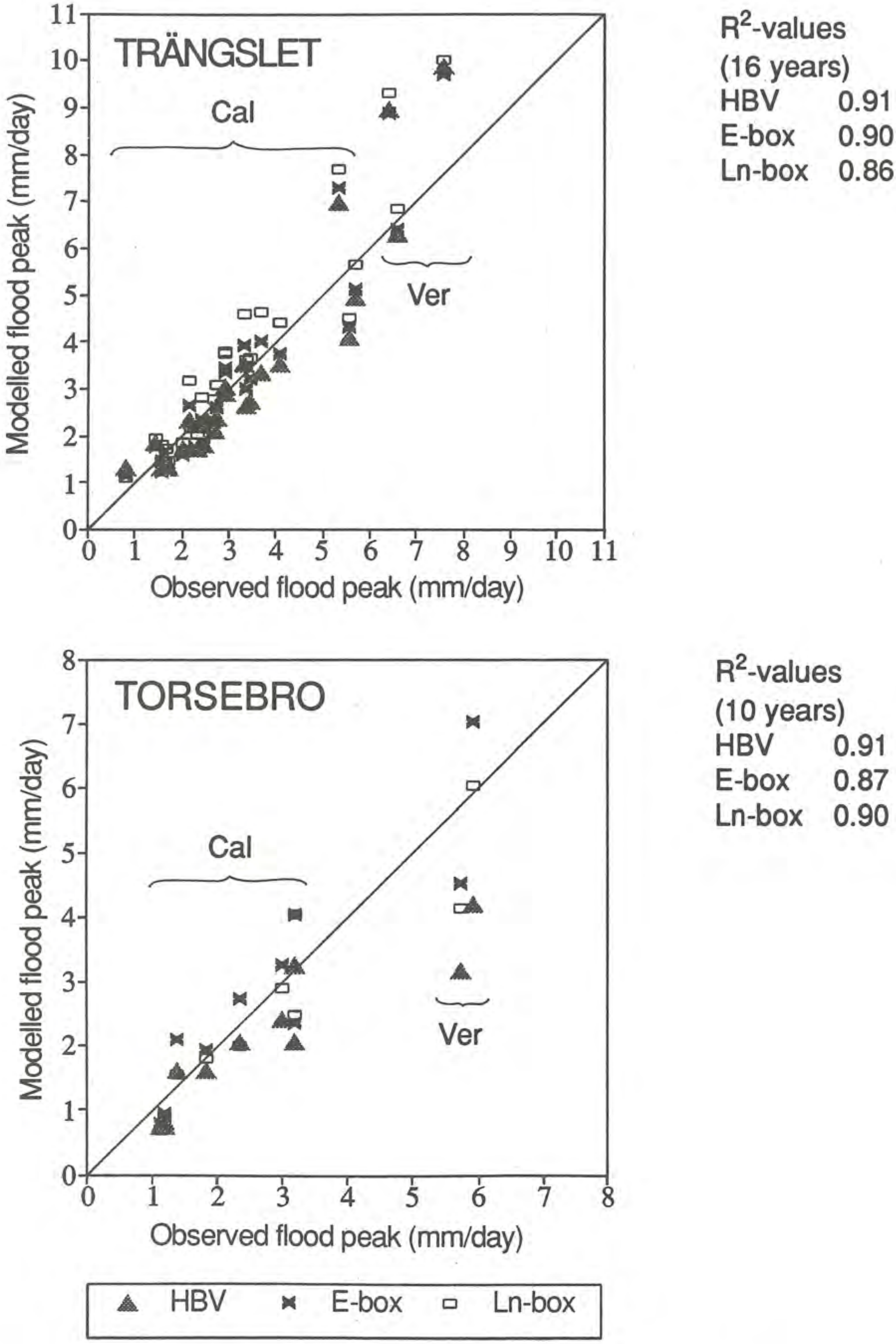


Figure 8. *Model performance for the tested runoff-response functions at Trängslet and Torsebro. The functions were calibrated and verified on rain floods only. R^2 values refer to calibration periods.*

The verification flood for Alfta (Fig. 7) is one of the best known extreme floods in Sweden. It culminated the 11th of September 1985, with a peak flow of 360 (m^3/s) at Alfta in the Voxnan river (Figure 9).

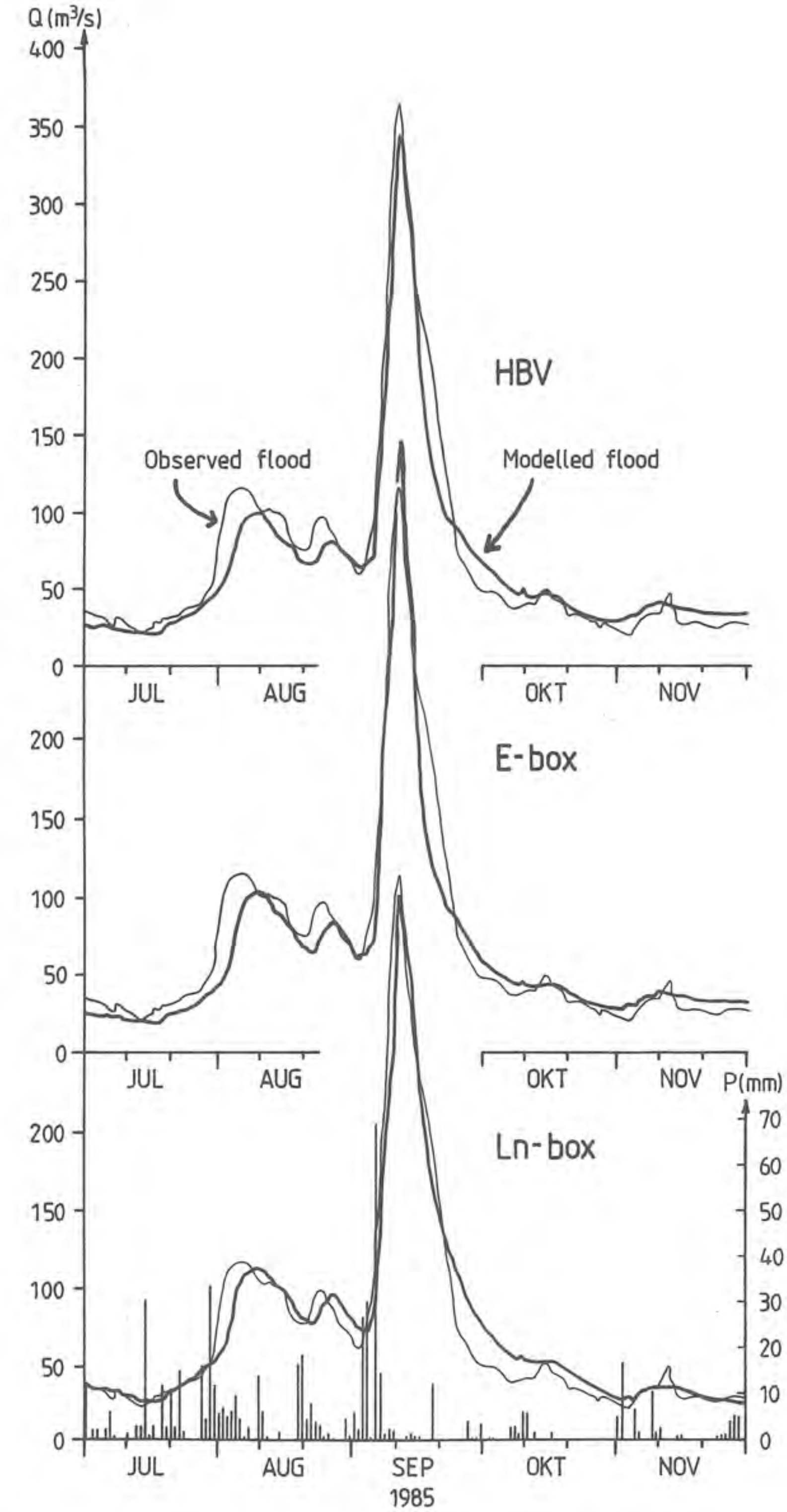


Figure 9. Modelled (thick line) and observed (thin line) runoff at Alfta during the extreme rain flood in September 1985.

The September 1985 flood was caused by extreme rainfall over the provinces of Dalarna and Hälsingland in Central Sweden. During July and August rainfall was larger than normal which resulted in low soil moisture deficits and well filled reservoirs. The heavy rainfall prevailed in the beginning of September with the largest rainfall occurring the 6th of September. On the 6th of September about 70 (mm) of rainfall fell over the northeastern parts of Dalarna and a large part of Hälsingland. On the 7th of September an additional 10 (mm) of rainfall fell. Large floods resulted, mainly in the Ore and the Voxnan rivers. The resulting flooding was the largest observed for the past 100 years.

The most spectacular consequence of this flood situation was an overtopping of the Noppikoski embankment dam in the Ore river. In the early morning of September 7, the dam eroded and within 45 minutes the 1 000 000 m^3 of water stored in the reservoir emptied out. During the dam failure the total outflow was about 600 (m^3/s). Luckily no people were injured or killed but severe damage was caused to several bridges and to the down stream forests (Kommittén för undersökning av allvarliga olyckshändelser, 1987).

Simulation of design floods

The uncertainty in design flood simulation, due to model structure, was estimated by simulating the most critical autumn floods (after August 1st) according to the Swedish guidelines for design flood determination (Flödeskommittén, 1990). Before the simulations, each response function was fine tuned over the largest of the observed rain floods. During the design flood simulation the soil moisture deficit was removed before the onset of the spring flood and the observed precipitation was replaced by a 14-day design areal rainfall sequence at the most critical location in time. The resulting peak flows are given in Table 3.

Table 3. Design flood simulation of the most critical autumn floods, using the HBV model with three different runoff-response functions. The table gives the resulting peak flow values (mm/day). The range is calculated with reference to the mean of the highest and the lowest peak value.

	Torrön (1983-10-16)	Alfta (1985-09-24)	Trängslet (1985-08-18)	Torsebro (1988-08-16)
HBV	63.3	14.2	19.6	10.1
E-box	113.1	18.5	31.8	14.3
Ln-box	91.5	16.6	25.1	13.0
Range	± 28 %	± 13 %	± 24 %	± 17 %

DISCUSSION AND CONCLUSIONS

The recession analyses were done for undisturbed rain flood recessions, that is periods where no rainfall was recorded during the recessions. These periods are however not truly undisturbed. Rainfall can have occurred over parts of the catchment where no gauge is located. The assumption that evapotranspiration could be neglected is also a weakness with the approach. However, only flood period recessions were examined, not the base flow recessions. This means that the evaporation will be negligible.

It was found that the runoff-response function of the HBV model could be substituted with a great number of different equations without radically changing the model performance. However, the results from the experiments carried out in this study indicate that none of the alternative runoff-response functions clearly performed better than the original HBV model function. An advantage with the original HBV model response function is that the effect of each parameter is easy to identify and calibrate. The HBV model gave throughout the best agreement over calibration periods, in particular for base flow periods. The largest disadvantage with the HBV model runoff-response function is that the parameter number is fairly large (five) and that the highest recession rate (K_0) is locked by the L_{uz} storage threshold. Of this follows, that simulation of larger floods than those used in calibration will have the same recession as the largest floods of the calibration period.

In general the E-box response function generated the largest flood peak followed by the Ln-box and the HBV. The differences between the models increased when simulating design floods. In terms of simplicity of application the E-box and HBV were easier to calibrate than the Ln-box. This was due to that the parameters of this function are strongly interdependent and that it was difficult to match the model to both base flow and flood periods with good results. In the Ljusnedal catchment this function performed poorly over base flow periods. It should however be borne in mind that the Ln-box is a single tank formulation with only two free parameters, which limits the degrees of freedom in calibration.

The E-box and the original HBV runoff-response function were found to be almost equivalent in application. The exponential increase in recession rate of the E-box could, however, possibly overestimate the recession rate of extreme floods. But this would require further research to be verified. If a runoff-response function with few parameters and with the ability of describing both floods and base flow periods is sought for, the E-box should be considered.

In formulating alternative response functions, the ambition was to reduce the number of free parameters and to formulate the equations so that the recession rate would increase with storage and flood magnitude. It was initially believed that such functions would be easier to calibrate correctly for extreme floods simulation since the equations would be active over all parts of the hydrograph. However, the results from calibration on small to moderate floods and verification over large observed floods (Fig. 7 and 8) indicate that it is difficult to select any response function as significantly better than the others. Furthermore, these figures show that the quality of the model performance over verification periods is linked to that over the calibration periods. It was also seen that for almost all of the verification period floods, the different response function formulations performed mutually similar. Either they all underestimated a certain flood or they overestimated the flood. The errors are therefore not only due to this model component but also to errors in model input and in the soil moisture routine. Another problem is that during extreme floods the rating curves have usually been extrapolated far beyond the range supported by direct measurements.

For the Torrön basin there occurred some snow accumulation during the largest flood, which also affects the results. Another problem in modelling the runoff from this catchment is that it has a fast runoff response. If for example, heavy rainfall starts in the evening one day and ends at midday the next day it will be split over two time steps (the time step used was 24 hours) in the model and thereby generate a smaller flood than the one observed. Design flood simulation according to the Swedish procedures avoid these problems, since all meteorological and hydrological inputs are preset on a daily time step. The Swedish guidelines for design flood determination state that a hydrological model with documented performance should be used in the simulations. In Sweden this means that the HBV model or similar is accepted. In Norway a simplified version of the HBV model is used for extreme flood simulation (Andersen et al., 1983). But the problem of how to assess the model error when extrapolating to such extremes as the design floods remains unsolved. This paper is an attempt to illustrate this uncertainty. The design flood simulations (Table 3) indicate that the modelling uncertainty is in the range of ± 20 % on an average. But nevertheless, further research focused on this problem is necessary before more precise estimates can be given.

The results showed that there is no clear tendency that the HBV model systematically underestimates extreme floods beyond the period of record. Therefore, as long as calibration is carried out with emphasis on the largest observed floods, the HBV model is appropriate for design flood simulations.

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VII

Proposed Swedish spillway design guidelines compared with historical flood marks at Lake Siljan

by

J. Harlin

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Proposed Swedish Spillway Design Guidelines Compared with Historical Flood Marks at Lake Siljan

Paper presented at ICWRS-Workshop on Risk
and Uncertainty in Hydrologic Design
Oslo, Norway, February – 1989

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A comparison between the proposed Swedish spillway design floods and historic flood marks made at lake Siljan in central Sweden, is shown. Frequency analysis is performed incorporating pregauge information on water levels together with a sensitivity analysis of modelling assumptions.

A water level of 0.42 to 0.75 metres above the highest historic flood mark (166.10 m.a.sl., 1659) was obtained when routing the design spring flood through lake Siljan. The design autumn flood lifted the lake to 1.56 to 1.52 metres below the highest flood mark. Return period for the design spring and autumn flood was estimated to about 1,000 years. The uncertainty in frequency analysis proved to have larger impact than modelling assumptions on estimating the risk of the design flood.

Introduction

New spillway design guidelines for Swedish dams have been proposed, Swedish Committee on Spillway Design (1989). The method suggested is based on a probable maximum flood (PMF) concept, employing a conceptual rainfall-runoff model in computing the design flood. This method has the advantage of combining experienced hydrological extreme events and giving not only the flood peak, but the whole design flood hydrograph. Hydrological modelling will introduce uncertainty, though in the model itself and in design assumptions. Because of the deterministic methodology, no probabilistic interpretation of the result is given. The classical problem of estimating the design flood return period has thus come in focus again.

One way to tackle this problem is by frequency analysis at a gauged site given a systematic flow record. However, only limited information about the flood frequency distribution at the site is obtained because of the limited experience the flow record reflects. Bergström, Lindström and Sanner (1989), found that the proposed Swedish spillway design guidelines represented floods with a return period beyond 10,000 years, on average over Sweden. To obtain more precise estimates of such extreme floods, one must bring additional information to bear on the problem.

Historic flood data such as high water marks occurring before the period of continuous data is one example of additional information. Although the number of recorded floods only increases marginally, there is also knowledge about the intervening years when no systematic record is available. Maximum annual floods during these periods were less than the historic floods whose values are known. Such a record can be considered as a censored sample from a frequency distribution and because of the historic longer period, improve the accuracy of the frequency analysis.

Benson (1950), was early to discuss the use of historic flood data. He studied a 72-year systematic record from the Susquehanna River at Harrisburg, USA, and a 88-year period prior to gauging with 7 historic floods. His work resulted in new plotting position formulas for the systematic and historical floods so as to remove the discontinuity in the frequency curve at the joint between the two sets of data. USWRC (1982), presents a method of moments technique for estimating distribution parameters with historic data. Also maximum likelihood estimators have been developed for several distribution functions, for example by Leese (1973), Stedinger and Chon (1986) and Clondie and Lee (1982). A comparison of the efficiency between different estimating techniques is presented by Chon and Stedinger (1987).

These papers all show that historic data give valuable information in describing the tail of the frequency distribution and increase the accuracy of the parameter estimates. Although this technique will not enable calculation of the probability of PMF-scale floods, U.S. Department of Commerce (1986), it will perform better than traditional frequency analysis and decrease the error bounds.

This paper addresses the question of how the proposed Swedish design floods compare with the extreme historical flood marks made at lake Siljan in central Sweden. It also attempts to estimate the return period of the design flood and analyze the sensitivity of different modelling assumptions.

Frequency Analysis

The frequency analysis is based on annual maximum water levels for lake Siljan. A continuous series of measured values from the unregulated period starting in 1887

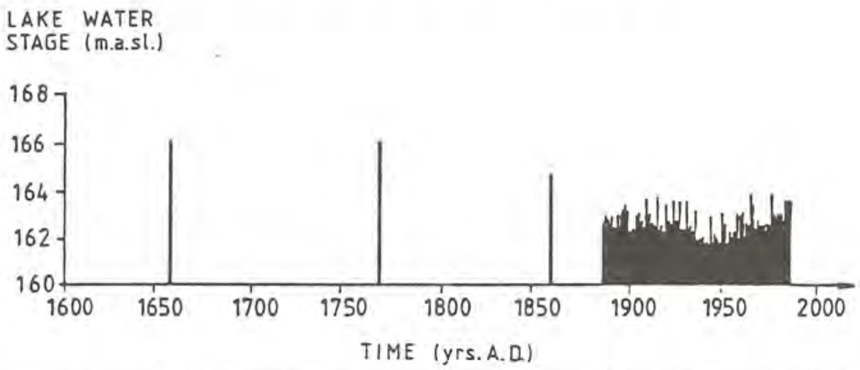

Fig. 1. Historical and systematically recorded annual maximum water levels of lake Siljan.

to 1926 is utilized. From 1927 to 1986 natural conditions have been reconstructed. In this way a continous series of 100 years is obtained.

Three historic floods are also incorporated in the analysis. These are the spring floods of 1659, 1764 and 1860. Fig. 1 shows the water stage data available for the frequency analysis.

The 1659 Year Flood

May the 8th 1659 the large flood caused the river to force a 1.3 kilometer long new channel close to the inlet to lake Siljan and destroyed Säbbenbo farm. This event has been documented by a number of authors in the 18th century and also in the church diary (Lannerbro 1953).

It is not exactly clear what water level the 1659 year flood caused. In a document concerning the regulation of lake Siljan, from 1915, the water level 166.10 m.a.sl. is given, which corresponds to the level given by some of the 18th century authors Lannerbro (1953). Wenner and Lannerbro (1952), who studied the meander field, geology and history of the Siljan basin question that the water could have risen that high, because a parish meeting report from May 19th 1661, describes the 1661 year flood as the lagest flood since 1598, and that larger erosion damage should have occurred.

A mark made at the steamboat pier in Mora, a town on the northen Siljan shoreline, gives the water level to 166.10 m.a.sl. and this is the level used in the analysis.

The 1764 Year Flood

Axel Wallén (1930) dates the flood to about the 20th of May. From church diaries at Hedemora, about 100 km downstream of Siljan, we know that large damage was caused by the flood which was considered as the worst since 1544 in this part of the river. The flood is also recognised by Wenner and Lannerbro (1952), and Lannerbro (1953) as an extreme flood, but no details are given.

A flood mark at the pier in Mora gives the maximum water level to 166.04 m.a.sl. and the flood is also marked as the highest historic flood on a rock foundation of the Kopparvågen house in Falun, some 50 km downstream. The 166.04 level is used in the analysis.

The 1860 Year Flood

This is the best documented of the historic floods. Wallén (1930) gives it the level of 164.70 m.a.sl. and places the flood as the highest in modern time and shows a photograph from 1909 of a flood mark carved on a house in Mora. Wenner and Lannerbro (1952), and Lannerbro (1953) also describe the flood, and flood marks are made both at the pier in Mora and on the Kopparvågen house in Falun. The Swedish Meteorological and Hydrological Institute SMHI (1966) give the maximum water level to between 164.90 and 165.00 m.a.sl. A level of 164.95 m.a.sl. is used in the frequency analysis.

To select an adequate distribution function five different frequency distributions were tried, namely the normal, lognormal 2, lognormal 3, Weibull and the Gumbel distributions. These were plotted in a frequency diagram together with probability plotting of the data set. The Chi-square goodness of fit test was performed. For spring water stage data, all but the Gumbel distribution were rejected at the 95 per cent confidence level. For autumn data the test could not reject any distribution.

Such tests are insensitive to distribution tail behaviour, which are the most important in this study. They also suffer from the weakness that the sample is used twice, once to fit the distribution and once to test the fitness. Cunnane (1985) concludes that »distribution choice cannot be based on theoretical arguments alone«. The Gumbel distribution was selected because of the better correspondence to the historical floods and its theoretical base as an extreme value distribution.

The equations for adjusting statistics for historic data defined in USWRC (1982) were used to estimate the distribution parameters. Plotting positions for the partially censored water stage record from the Bayesian plotting position formula derived by Hirsch and Stedinger (1987) were used.

Design Flood Modelling

The Swedish Committee on Spillway Design (1989) describes the method used in detail. A summery of the proposed Swedish spillway design guidelines is found in Bergström *et. al.* (1989). To apply it to the Siljan basin the HBV model, Bergström (1976), was calibrated for five separate subcatchments and linked together. The subcatchment outlets were at the outlets of lake Idre and lake Skattungen, furthermore at Trängslet and Bössbo, Fig. 2. The catchment area at the main outlet in

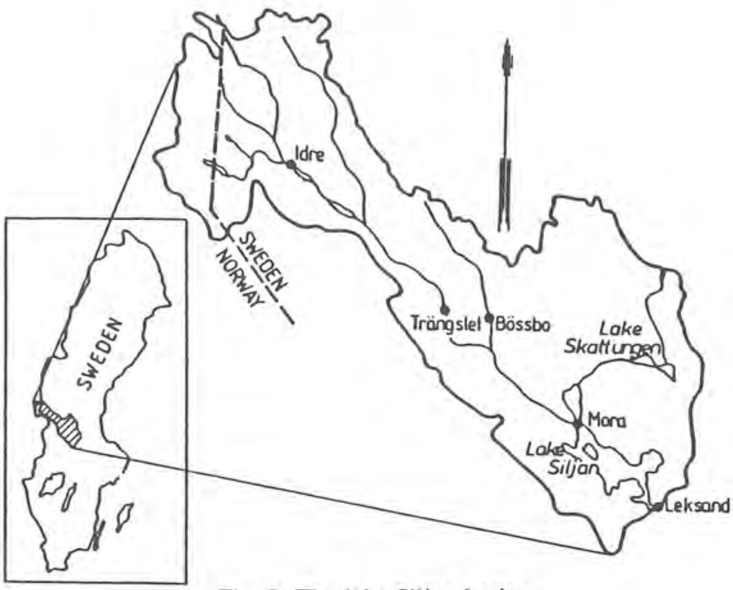

Fig. 2. The lake Siljan basin.

Leksand is 11,973 km^2 of which the lake Siljan constitutes 289 km^2 at average filling.

The Idre and Skattungen lakes were modelled separately with original stage discharge relations. A critical part in the simulation was to describe the outflow from lake Siljan, in particular at high water levels.

The Stage-Discharge Relation of Lake Siljan

The outflow of the lake Siljan is controlled by a natural section control, caused by a gradually increasing slope and constricting width of the outlet channel. This was described by the simple stage-discharge curve. Twentyfive discharge measurements, between the years 1903 and 1916 at different discharge rates have been made at the outlet. These were used to compute the constants in the stage-discharge equation. According to Cook (1987), the theoretically correct rating curve will have a slope less than 2.0, usually between 1.3 and 2.0. The found slope of 1.576 conforms to these theoretical considerations.

The stage-discharge relation should be more accurate than any of the individual gaugings. To get an estimate of the uncertainty of the spread or dispersion of the gaugings about the stage-discharge relation the standard error of estimate *Se* according to WMO (1980), was calculated. Provided that there is no change in the control or the hydraulic conditions, 19 out of 20 of all current meter observations should be included within these limits. At the Siljan outlet 23 of 25 current meter observations were well within the *Se* limits and two were right on the limit boundary.

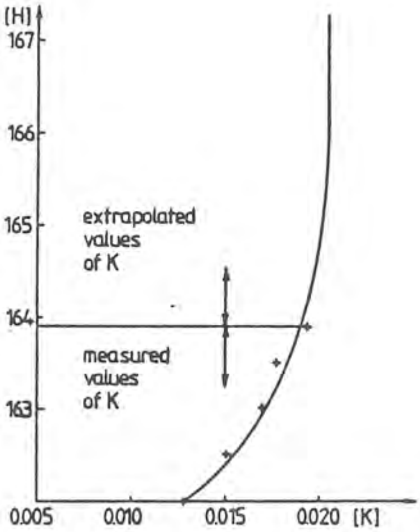

Fig. 3.
Extrapolation of the conveyance coefficient K at high water levels of lake Siljan.

Uncertainty in the stage discharge relation, expressed as a percentage, given by the standard error of the mean *Smr* (WMO 1980), was computed. *Smr* values varied between ± 2 % at intermediate water levels to ± 3 % at high levels of lake Siljan.

Extrapolation of the Rating Curve

The highest historical flood mark at lake Siljan is at an elevation of 166.10 m.a.sl. This should be compared to 163.90, which is the level at the highest current meter observation. An extrapolation of 2.2 metres has thus to be made to be able to estimate the magnitude of the historical floods. The design flood is in this order of size which is the reason for extrapolating the rating curve.

Assuming that the channel cross section and roughness has not changed with time and that the effective control during the historical flood events were the same as the effective control at the upper range of the current meter measurements, an extrapolation can be made. The method chosen was the slope conveyance method (Dalrymple 1948), which is based on application of the Manning equation.

At the lake Siljan outlet the cross section is fairly regular and no overbank flow occurs within the extrapolation range. A plot with stage as the ordinate and the mean velocity as the abscissa gave a curve which tended to become asymptotic to the vertical at higher stages. Because the rate of increase in the velocity at the higher stages diminishes rapidly, this curve could be extended without much error.

For high water stages, the Manning equation can be rewritten as

$$Q = KAR^{2/3} \tag{1}$$

$$v = KR^{2/3} \tag{2}$$

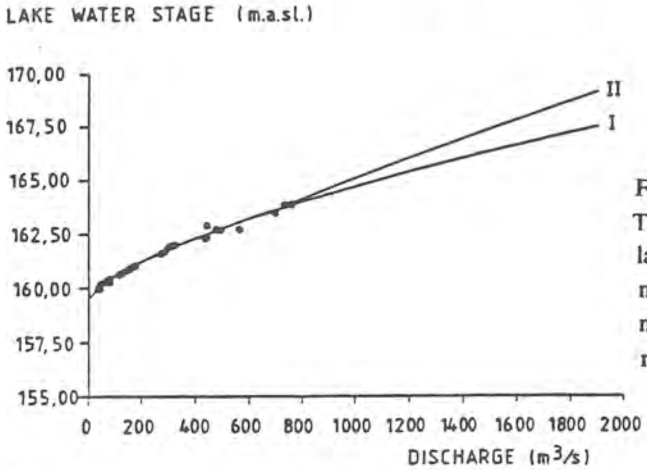

Fig. 4.
The stage discharge relation of lake Siljan. Extrapolation II is made by the slope conveyance method, curve I is the extended rating curve.

Values of the conveyance K were computed by using various values of υ from the known upper portion of the rating curve and the corresponding values of R. These values of K were plotted against gauge height, Fig. 3. This curve was extrapolated and high stage values were combined with their respective values of A and R to give the discharge.

The resulting stage discharge curve extrapolated by slope conveyance and by extending the original rating curve to higher levels is shown in Fig. 4.

Results

Spring Flood

The spillway design spring flood caused lake Siljan to rise to an elevation of 166.52 or 166.85 m.a.sl. depending on which extrapolation method of the rating curve used. The higher level corresponds to the slope conveyance extrapolated rating curve.

In terms of frequency this corresponds to 3,450 and 6,495 year return periods if the analysis is based on the systematic period of annual maximum water level observations, Fig. 5.

When the three historic floods are incorporated in the frequency analysis the return periods drop to 590 and 965 years respectively, Fig. 6.

The Swedish spillway design guidelines prescribe a snowpack with a return period of 30 years according to the Gumbel distribution. Resulting water levels and return periods, at different snowpacks were simulated. Rating curve extrapolation by the slope conveyance method was chosen and the frequency analysis incorporated the historical flood marks. A straight line in semi-logarithmic scale fitted by least squares was drawn, Fig. 7.

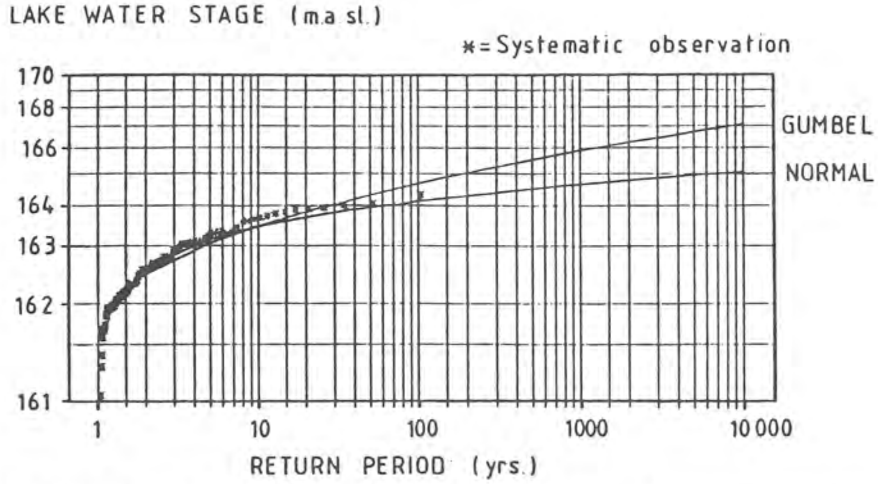

Fig. 5. Frequency analysis based on systematic recordings of annual maximum water levels. The Gumbel distribution was used and for reference the normal distribution is depicted.

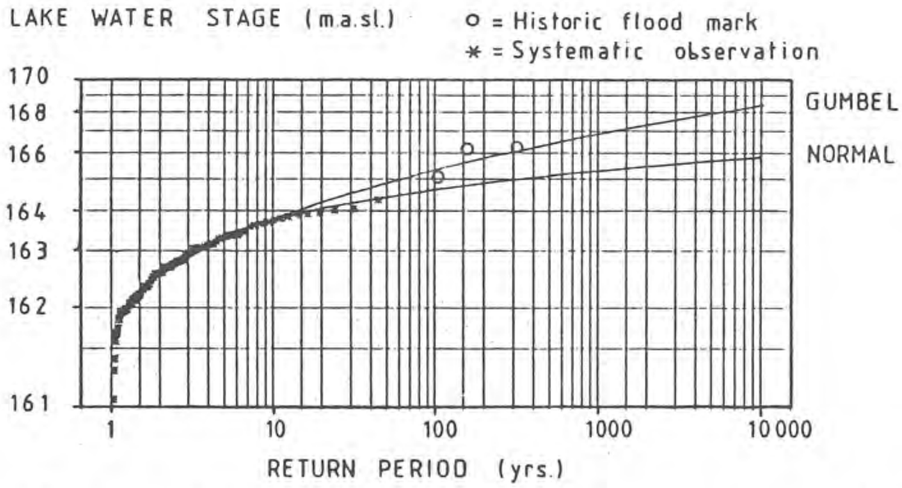

Fig. 6. Frequency analysis based on systematic recordings of annual maximum water levels and historical flood marks. The Gumbel distribution was used and for reference the normal distribution is depicted.

Autumn Flood

Elevations of 164.54 or 164.58 m.a.sl. were the results of routing the design autumn flood through lake Siljan. As initial water stage the prescribed mean maximum autumn water level was employed.

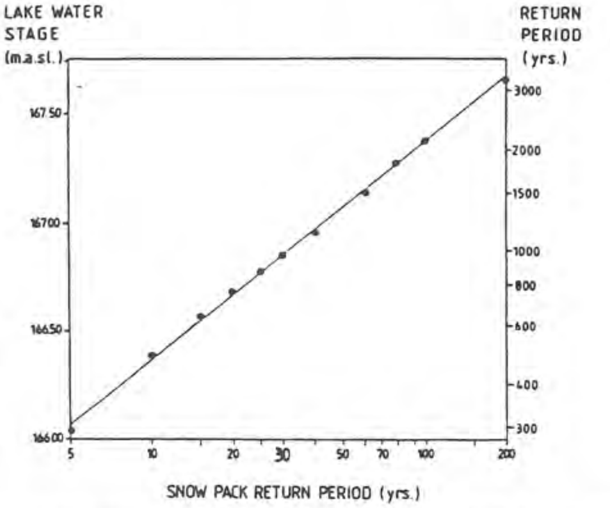

Fig. 7. Lake Siljan water stage and corresponding return period at different snowpacks.

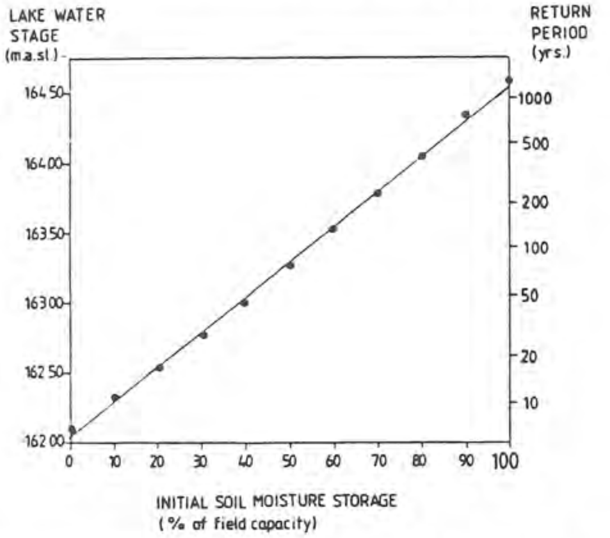

Fig. 8. Lake Siljan water stage and corresponding return period at different initial soil moisture storage.

No historic flood marks for autumn floods have been made. Frequency analysis using the Gumbel distribution, 50 years of observed and 37 years of reconstructed maximum autumn water levels, resulted in return periods for the design autumn flood at 1,180 and 1,280 years respectively.

Initial soil moisture storage is prescribed to 100 % of field capacity. The sensitivity of this prescription was tested and a least squares regression fitted, Fig. 8.

Discussion

The proposed Swedish design spring flood proves to yield water levels of about half a metre above the extreme historic flood marks. If such a climatic and hydrologic situation would occur it would give rise to an increase of the lake Siljan water level, disregarding the effects of regulation, of about 6 m. In a normal year the spring flood would lift the lake about 2 m. Today much of the hydropower potential has been utilized and several dams have been constructed, *eg.* the Trängslet dam. These reservoirs would probably store part of the flood and damp out the flood peak and thereby reduce the lifting of the lake Siljan.

The design spring flood is computed by repeated simulations of different snowmelt scenarios where the design precipitation sequence is tried at alternative dates until the most critical combination is found. At Siljan this combination was in general when the precipitation sequence was placed just at the end of the melting period. Melt water had then raised the lake and together with the precipitation during the most critical days, an extreme combination was generated. Maximum 24-hour effective precipitation for the design spring flood was 65.3 mm where 2.8 mm was contribution from snowmelt.

At snowpack return periods exceeding about 50 years the snowmelt contribution had a more important role and the design combination was often during the most intensive snowmelt.

The proposed design autumn flood was not able to lift lake Siljan to the extreme levels of the historic flood marks. These marks are made however, at spring floods and therefore not directly comparable with autumn conditions. The autumn flood was generated by the 14-day design precipitation sequence with a maximum 24-hour value of 93.3 mm.

In autumn the choice of initial water level was crucial for the calculation. At spring there is a payoff between snowpack and water stage but at autumn the water level is a result of summer and early autumn rains. For comparison the autumn simulation was made starting at the highest recorded autumn water level (163.03 m.a.sl. 1985). This simulation gave the results 165.31 or 165.41 m.a.sl., the latter using the slope conveyance extrapolation, which is about one metre above the design level.

It is surprising that the design spring flood water stage only exceeded the highest observed level in the past threehundred and thirty years by half a metre. Consequently the return periods of the design floods were much lower than expected. The return period for both the spring and the autumn design flood was estimated to about 1,000 years opposed to beyond 10,000 years, concluded by Bergström *et.al.* (1989). This could possibly depend on that their analysis mainly concerned streamflow, while this study dealt with water levels of a lake with a large damping effect on inflow, and a long hydrological memory.

However, the frequency analysis was very sensitive to choice of correct frequency distribution and data type and did not give confident estimates of floods of such low probability as the spillway design floods. Return period differences between frequency distributions was in the order of ten to threehundered times. The second largest source of uncertainly was whether the historical floods were incorporated or not. When the historic floods were utilized, the return periods dropped about ten times. Since it is always questionable how high such floods were and if there has been any change to the outlet of the lake Siljan over time, one has to be careful to rely heavily on these old recordings. Uncertainty in modelling the outflow from lake Siljan had the least influence on return period estimation. This uncertainty was in the order of two times, on the design flood return period.

To tackle the classical problem of estimating the return period of design floods, in the new context where the floods are created by PMF type of methods, proves to be an extremely difficult task. Often, extrapolation to roughly hundered times the record length has to be done. One has certainly to bring additional information to the analysis to enable this.

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I thank Mr. Anders Heldemar at the River Dalälven Regulation Enterprise for initating the project, making data available for the study and for valuable discussions. I also want to express my gratitude to my colleagues at SMHI for help and advice, in particular Mr. Göran Lindström and Prof. Sten Bergström.

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