Institutionen för vattenbyggnad
Chalmers Tekniska Högskola

Department of Hydraulics
Chalmers University of Technology

Rainfall Data for the Design of Sewer Pipe Systems

Viktor Arnell

Report
Series A:8
ISSN 0348–1050

Göteborg 1982

Address: Department of Hydraulics
Chalmers University of Technology
S-412 96 Göteborg, Sweden
Telephone: 031/81 01 00

PREFACE

The studies on rainfall data for the design of sewer systems is a part of the research on urban hydrology at Chalmers University of Technology. Results obtained with a model for the design of storm-sewer systems, rough estimates of volumes and peak-flows, storm-water pollutants, and pipe-flow routing have been presented earlier. Work is continuing on technical-economic risk-based design of sewers, overland flow and discretization of runoff basins, and energy losses at storm-drain junctions.

Earlier reports on rainfall data deal with the evaluation of new Intensity-Duration-Frequency curves for Göteborg (5 stations, 45 years of data, Arnell 1974) and relationships for the estimation of overflow volumes and overflow durations (1 station, 18 years of data, Arnell and Asp 1979). The research on rainfall data will continue with studies of rainfall data for the design of retention basins and for the estimation of overflow volumes.

The work was financially supported by the Swedish Council for Building Research (project No. 780257-8), Chalmers University of Technology, and the local Water and Sewage Works in Göteborg.

Göteborg in November 1981.

Viktor Arnell

ACKNOWLEDGMENT

During the rainfall studies described in this report, Professor Anders Sjöberg acted as a supervisor and contributed with much valuable advice.

Thomas Asp and Bo Ekelund have carried out the treatment of the precipitation data, and Håkan Strandner has made the computer programs for the estimation of the local design storms.

The transformation of the rainfall diagrams to the computer was done at the Swedish Meteorological and Hydrological Institute under the supervision of Bengt Dahlström.

Ebbe Ryberg, Leif Staberg, and Lars Svärd from the Water and Sewage Works at Göteborg have taken part in the project.

Alicja Janiszewska has made a great number of the runoff simulations and has drawn the figures. The manuscript was typed by May-Britt Fryksmark, and Gerd Eng has corrected the language.

I wish to express my sincere thanks to all the persons mentioned above and to others who have helped me carry out this work.

Viktor Arnell

LIST OF CONTENTS Page

SUMMARY

This report contains a comparison of designs of sewer pipes using different types of rainfall data, and the aim is to enable the reader to select correct rainfall data for the design of sewer-pipe systems in different applications.

Two different kinds of rainfall data can be used: Design storms based on statistical values and Historical storms obtained through measurements. A design storm is a rainfall which is developed for a certain design return period, and the flow value which is calculated by means of the storm is said to obtain the same return period as the storm. When historical storms are used, a number of storms are run through a runoff model, and the statistical analysis is applied to the simulated flow values to find the flow value coupled to the design return period.

As a design criterion, the one applied today has been used: The water level is not allowed to reach the head of the pipe more frequently than the chosen return period. To make possible a statistical analysis, independent flooding events were defined by autocorrelation analysis of successive rain volumes for specified time intervals. It was found that the rainfall values are independent after on the average 4 hours or longer. For the study described in this report an 18-year record (1921-1939) of rainfall data for Lundby, Göteborg, is utilized.

Most design storms are connected with the Intensity-Duration-Frequency (I-D-F) relationships. When they are estimated, maximum average intensities for different durations are searched for each storm. Each duration is treated separately, and statistical distributions are estimated for each duration. The different points of the resulting I-D-F curve may thus come from different historical storms, as one curve is valid for one return period and the data come from different durations. I-D-F curves for durations of from 5 minutes to 240 minutes

have been estimated for Lundby, Göteborg, for rainfall events separated in time by rain-free periods of 4 hours or longer. The mathematical log-Pearson Type III distribution was fitted to the data, and to the resulting I-D-F curves were fitted mathematical expressions of the form $i_m = a/(T+b) + c$, where i_m is the maximum average intensity for the duration T and a, b, and c are constants different for each return period.

The following design storms have been tested: *The Average-Intensity-Duration (I-D-F) design storm,* which is just the constant intensity obtained from the I-D-F curves. The volumes of the I-D-F design storms vary between 35% and 75% of the volumes of the historical storms. *The Chicago design storm,* which is determined from the mathematical expression of the I-D-F curve and whose most important characteristic is that the maximum average intensities for all durations follow an I-D-F curve. *The Sifalda design storm,* which is compounded of the I-D-F design storm with precipitation added before and after, giving the Sifalda storm a total volume similar to the total volumes of the historical rainfalls. *The Illinois State Water Survey (ISWS) design storm,* which consists of the rain volume obtained from the I-D-F curves and with a time distribution equal to the average distribution for the historical storms. *The Flood Studies Report (FSR) design storm,* which has a total volume obtained from the I-D-F curve and with a time distribution equal to the average distribution for the historical storms centered around the most intense parts. Local coefficients for the different design storms were evaluated from the data for Lundby, Göteborg.

Previous investigations of the use of historical design storms have dealt with the screening of continuous rainfall records to find a smaller group that can be used for the final runoff simulations. The most common method has been to select the rainfalls with maximum average intensities larger than specified threshold values for different durations and in some cases combined with a volume criterion. One method has been to use a simple and thus inexpensive method (e.g. the linear unit-hydrograph method) to select the historical rainfalls corresponding to the design return period.

Historical rainfalls as well as the earlier mentioned design storms have been used for the calculation of peak-flow values in the pipe systems in each of three test catchments of the sizes 0.154 km^2, 1.450 km^2, and 0.185 km^2. The sewer systems have a tree-like structure, and 7 design points were chosen in each basin for the comparison of the peak flows for different rainfall data. No runoff was assumed to come from the permeable areas, and thus the runoff also was assumed not to be influenced by antecedent precipitation. These assumptions were verified through rainfall-runoff measurements. The runoff calculations were made with the CTH-Model, which is a single-event model including the processes of infiltration, surface depression storage, overland flow, gutter flow, pipe flow, and retention storage. The model has previously been validated in the catchments used in this study.

Peak flows were calculated for the return periods of 1/2, 1, 2, and 5 years for the different design points and for each typ of design storm. For each design storm, except the Chicago design storm, different durations of the part of the storm obtained from the I-D-F curves were applied to find the duration that caused the maximum peak-flow value. For the most upstream parts of the basins it was found necessary not to vary the durations more than 1-2 minutes to find the maximum peak-flow values.

Peak flows also for the heaviest historical storms were calculated and plotted on statistical probability paper for return periods of 1/2 year and longer. The results of the calculations for the design storms were plotted in the same diagram. The runoff calculations for the historical storms were made for the rainfalls contributing to the 54 largest maximum average intensity values for durations of from 5 minutes to 30 minutes for the smaller basins and to 120 minutes for the large basin. In this screening of the 18-year rainfall series, it was found possible to include the return period of the peak flows of 1/2 year.

"Historical design storms" were selected by the simple unit-hydrograph method, where the unit hydrographs were estimated with the CTH-Model for constant rainfall intensities corresponding to a return period of one year and a duration of 10 minutes for the smaller basins and 20 minutes for the large basin. Historical design storms common for the design points were identified by accepting a deviation of $\pm$ 5% of the peak-flow value corresponding to the design return period at each design point. For several return periods, one or two historical storms could be used for the final design for the entire catchments.

The estimations of design peak flows made with the CTH-Model and all historical storms were assumed to be the most correct estimates when the designs for the different types of rainfall data were assessed, and the results for the other storms were expressed as over- and underestimations compared to those estimates. Besides, it was found that the over- and underestimations as well as the standard deviations of the over- and underestimations should be as small as possible, but uncertainties (standard deviations) of 10-15% can be accepted without large increases in the total uncertainties. Overestimations of up to 10% can be accepted for small basins without involving too large extra investment costs. For large basins even small overestimations cause significant extra investment costs. Changes in peak-flow values of more than 10% give significant changes in the return periods for the peak flows.

Compared with the design peak flows estimated for the historical storms, the best results for the design storms were obtained for the Sifalda design storm, or on the average an underestimation of the peak flows of 2% and a standard deviation of 4%. The I-D-F design storm gave underestimations of on the average 9%, which result is supported by similar results found by other researchers. The Chicago design storm, with its high peak, caused overestimations of on the average 5%, which is less than expected. The ISWS storm gave underestimations of on the average 6%, which could be expected because the local ISWS storm is similar to the I-D-F storm. The local FSR design storm, which has an unrealistically high peak, caused large overestimations of the peak flows, that could not be explained. Further studies of the FSR storm are needed.

Concerning the use of historical storms, rather good results were obtained when designing by the unit-hydrograph method and using all historical storms. The overestimations were on the average 3%. The unit-hydrograph method was found to be very sensitive to the rain intensities used for the estimation of the unit hydrographs. The method can be improved if the relations between the intensities of the historical storms and the intensities used for estimations of the unit hydrographs are known. A number of unit hydrographs valid for different rain intensities must be estimated at each design point.

Good estimates of the design peak flows were obtained with the CTH-Model for the historical storms selected by the unit-hydrograph method. However, there is a risk for over- and underestimations if unsuitable historical design storms common for the design points are selected. This risk was studied by estimating the mean values plus/minus the standard deviations (MV $\pm$ σ) of the differences between the peak flows estimated with the CTH-Model, and belonging to the $\pm$ 5% group, and the design peak flows estimated for all historical storms. The values of MV $\pm$ σ were found to be 5 - 15% for the smaller Bergsjön and Linköping 2 basins and 5 - 30% for the larger Linköping 1 basin. The variations decreased only a little when the groups were reduced to $\pm$ 2.5% or when the unit hydrographs were estimated for another constant rainfall intensity. The significant over- and especially underestimations are due to the fact that the underlying assumption of the method is not valid. The assumption is that the rainfalls corresponding to the peak-flow values are ranked in the same order whether the peak flows are calculated by a unit hydrograph method or with a more detailed model.

The conclusions concerning rainfall data for the design of sewer-pipe systems are:

- The use of the Sifalda design storms gave the best results for the examples of design using design storms.
- The use of the I-D-F design storms and the ISWS design storms gave underestimated peak flows, and the Chicago design storms gave small overestimations.
- Acceptable results of designs by the unit-hydrograph method can be obtained if a correct rainfall intensity is used for the estimation of the unit hydrograph.
- Good results of designs were obtained in the examples for historical storms selected by the unit-hydrograph method. However, the risk for, especially, significant underestimations is obvious.
- The examples given in this report do not show that the use of design storms gives significantly worse designs than the use of historical storms as long as the interest is focused on the peak flows only.

To facilitate the use of correct rainfall data, it is of utmost importance to evaluate available rainfall data and to start new rainfall intensity measurements. Further research should be focused on the areal distribution of rainfall and how to take that distribution into consideration when using distributed runoff models, and on rainfall data for the design of retention basins, overflows, pumping stations, and the operation of sewer systems and treatment plants.

1. INTRODUCTION

1.1 Design of Storm Sewer Systems

The design of storm sewer systems is an optimization process, where the construction costs, the maintenance costs, and the flooding costs should be minimized within the constraints of rules and regulations and acceptable damages of floodings. Variables to be optimized are the lay-out of the pipe system, pipe diameters, pipe slopes, and sizes of retention basins, if any. Examples of constraints besides the mentioned above are minimum cover depth, minimum pipe diameter, and minimum pipe slope.

A number of numerical computer-based models have been developed for the optimization of sewer systems. Nilsdal (1981) has made a literature review of some of the existing models. Work is now in progress on the application of one model to Swedish conditions.

Some of the methods incorporate the concepts of risk and uncertainty in the applications of sewer system design (see, for example Tang, Mays, and Yen, 1975). The risk is defined as the probability of occurrence of a variable greater than the design magnitude during a specified period of time, or expressed by probability symbols

$$P(X>Q) = 1 - (1 - \frac{1}{F})^n \qquad \ldots(1.1)$$

where $P(X>Q)$ = probability of the flow X being greater than the design flow value Q

F = design return period

n = number of periods for which the risk is estimated, e.g. a number of years

Equation (1.1) gives the relation between the concept of return period and risk. The equation is difficult to apply directly to the design of ordinary sewer systems because n is large (~50 - 100 years) compared to F

(~1 - 5 years), which gives values of $P(X>Q)$ close to one. The approach is, however, promising and gives possibilities to include the uncertainties in various design models and their input data and to make a more correct design from a statistical point of view. The same approach is used in, for example, geotechnical engineering work and offshore engineering. Work is now in progress to see if the approach can be adapted to sewer system design.

Today the optimization of sewer systems is expressed by the choice of the design return period. It is believed that the chosen return period gives an optimal technical-economic design, which is acceptable from a social and legal point of view. In Sweden, the Swedish Water and Waste Water Works Association (VAV, 1976) has given recommendations for the selection of design return periods for the design storm. As a logical consequence, the design peak-flow value must obtain the same return period. For sewer pipes, the water level is not allowed to reach the head of the pipe more frequently than the chosen return period.

In this report the return-period approach is used because the risk-based design is not yet fully developed for sewer pipe design.

Two different kinds of rainfall data can be used for the design of sewer pipes:

- Design storms estimated from Intensity-Duration-Frequency relationships or from historical rainfall data

- Historical storms or time series generated by statistical methods.

A design storm is a rainfall which is developed for a certain design return period, and the flow value which is calculated by means of the storm is said to obtain the same return period as the storm.

When historical storms are used, a number of storms are run through a runoff model, and the statistical analysis is applied to the simulated flow values to find the flow value coupled with the design return period.

The conclusion is that when design storms are used, the design is based on rainfall statistics and when historical storms are used, the design is based on flow statistics. The latter must be more correct, since the flow values are the interesting design variables.

The design storms have the advantages of being easy and inexpensive to use. They are, however, evaluated for normal runoff areas only, and can give different results for different types of areas. The use of historical storms give more correct results, for all types of areas, and take into account non-linear effects in the runoff process that influence the flow statistics. Historical storms can be more expensive and complicated to use, but that problem can be minimized through well-developed manuals and computer routines.

When detailed mathematical runoff models were first utilized for the design of sewer systems, the only rainfall data used were different design storms. Later, the design storms were subjected to criticism due to their weak statistical properties, and the use of historical storms was suggested (see, for example McPherson, 1977). However, during the last few years the development of design storms for storm-water analysis has become more focused on the interesting runoff variables, such as values of peak flows and runoff volumes. Packman and Kidd (1980) and Urbonas (1979) have, for example, reported on the development of design storms that are expected to give correct information on design flow statistics.

The work reported in the present thesis began in 1975 based on ideas given by Professor M.B. McPherson in his discussion following the paper by Keifer and Chu (1957) and after personal communication with Professor J. Amorocho, the University of California at Davis. They felt that it should be possible to screen the continuous historical rainfall record and select the interesting storms that are important for the design of the sewer systems. These storms could then be used in the design work and the statistical analysis applied to the runoff variables.

Parallel to my work, studies have been going on both on the selection of historical storms and on the use of design storms, resulting in no clear evidence for which of the two types of storms is the best (see, for example Johansen and Harremoës, 1979, and Packman and Kidd, 1980). Therefore, it was decided to include both the use of design storms and the use of historical rainfalls in the present study. Even if the use of historical rainfalls is more correct, it seems likely that design storms can be used for some applications, especially as there are only a few historical rainfall records available i Sweden. A preparatory study was reported at the International Conference on "Urban Storm Drainage" at Southampton, England (see, Arnell, 1978). That study did not show any large differences in simulated peak flow values for design storms compared to values obtained for historical storms.

1.2 Scope of the Report

The aim of the work published in the present report is

o to enable the reader to select correct rainfall data for the design of sewer-pipe systems in different applications.

The report includes comparisons of designs of sewer-pipe systems using the two types of rainfall data, design storms and historical storms.

In Chapter 2 a description is given of the development of Intensity-Duration-Frequency (I-D-F) relationships, on which most design storms are based. I-D-F relationships have been developed for an 18-year historical rainfall record in Göteborg.

Different design storms are described in Chapter 3. These are the Average-Intensity-Duration storm, the Chicago design storm, the Sifalda design storm, the Illinois State Water Survey design storm, the English Flood Studies Report design storm, and a French design storm. For all but the French storm, local coefficients were estimated from the 18-year historical rainfall record. At the end of Chapter 3 a literature review is given of the use of design storms compared to the use of historical storms for the design of storm sewers.

A literature review of the use of historical rainfalls in the design of sewer systems is given in Chapter 4. Different methods for screening continuous records and selecting the most important storms are given.

The rainfall measurement station at Lundby, Göteborg, is described in Chapter 5. An 18-year historical rainfall record for the period of 1921-1939 was chosen for the investigation. The continuous record was divided into separate events, according to a definition of independent flooding events given in Section 1.3.

Chapter 6 contains a description of the design of sewer pipes for different design storms and for historical storms. Runoff simulations are also carried out with unit hydrographs estimated with the CTH-Model. Furthermore, the chapter includes a description of the test catchments and the CTH-Urban Runoff Model used in the study. (Further details of the model are given in Arnell, 1980.)

Designs based on different rainfall data are compared in Chapter 7, which also contains a discussion of errors and uncertainties involved. Effects on investment costs and return periods for flooding are treated. Designs for different design storms are compared with designs for all historical storms. Different methodologies using unit hydrographs in the design are analyzed.

Recommendations for practical applications and future research are given in Chapter 8.

The different design storms tested in the report were all obtained from literature. No attempt has been made to develop a special type of design storm for Swedish conditions.

The 18-year continuous record of historical rainfall data used in this investigation is one of the very few evaluated historical records in Sweden. The total amount of data available on computer tape in Sweden corresponds to 69 years of data for four places (Dahlström, 1979). Probably still more data are hidden at different places in the country. This is one of the reasons why the approach based on historical storms may not be usable at all places in Sweden. Design storms are the only alternative if one cannot transform a historical record from one place to another. Dahlström (1979) has correlated the Intensity-Duration-Frequency relationships with a measure of the amount of convective precipitation estimated as the difference in amounts of precipitation for a period with much convection and a period with little convection. Periods of one month were used and since there are monthly precipitation values for most parts of Sweden, it was possible for Dahlström to estimate the variation of the I-D-F curves for the whole country. This makes it possible to estimate values of design storms also for different parts of Sweden.

The investigation in this report is based on point precipitation only. The spatial variation in rainfall intensities can significantly affect the flow values for large areas. That fact is not included in this study, and it is assumed that it is of minor importance for the test catchments used, because of their small size (0.15, 0.19, and 1.45 km^2). Since knowledge is limited about how to include the spatial variation in time-varying rainfall data, at least for the near future, the design of sewer systems must be based on point precipitation.

1.3 Definition of a Flooding Event

The statistical analysis carried out when designing a storm sewer system should be applied to independent runoff peak flows. Thus, the different runoff peaks should be separated in time so one event is independent of surrounding events. To this statistical definition can be added sewage technology aspects. If repair and clean-up after one flooding have not taken place before the next one occurs, the two floodings should be considered as one flooding event. The time required for repairing and cleaning depends on the type of settlement, and what type of activities are going on in the flooded area. Because no such data are available, the independence between events is measured in a statistical way only. The factors that govern the necessary time between independent events are different for different runoff areas, which means that the time will be different for different areas. From practical points of view, it is an advantage to work with one or only a few different time intervals between independent events. The separation of a continuous rainfall record into independent events can, thus, be made once and for all.

The analysis of independence should be carried out on a time series of flow values, but no such series of sufficient length is available. To generate a long time series of flow values by a runoff model is possible but expensive. For this study, the statistical independence is therefore evaluated by analysis of a time series of rainfall data. The runoff response in urban areas is so fast that the result is assumed to be approximately the same if the analysis is carried out on precipitation data as if it were carried out on flow data. The time between two rainfalls should, however, be longer than the time of concentration to take into consideration some of the effects that are included when the rainfall is transformed into runoff. For a sewer system with retention basins, we must add the restriction that the time between two rainfalls must be longer than the time needed to empty the basin after the first rainfall.

The independence is evaluated by autocorrelation analysis. The correlation is evaluated between successive rain volumes for specified time intervals, where the intervals are delayed in time. The time lag is varied from zero, for which the correlation is unity, to a time lag for which the correlation is negligible. The autocorrelation analysis is applied to the continuous time series and not to the peak intensities only. This is because the peak-flow values are influenced by also the rain intensity values preceding the peak-flow values.

The autocorrelation analysis was carried out in the following way. Precipitation data for Lundby, Göteborg, for 1921-1939 (see Chapter 5) were transformed to rainfall depths for successive, equal time intervals, Δt. The correlation was then calculated between rainfall depths at different time lags by a standard computer program (IMSL FTAUTO), which uses the following equations (see also IMSL, 1975, and Fig. 1.1):

$$r_\tau = \frac{\text{cov}\ (p_t,\ p_{t+\tau})}{\text{var}\ p_t} \qquad \text{...(1.2)}$$

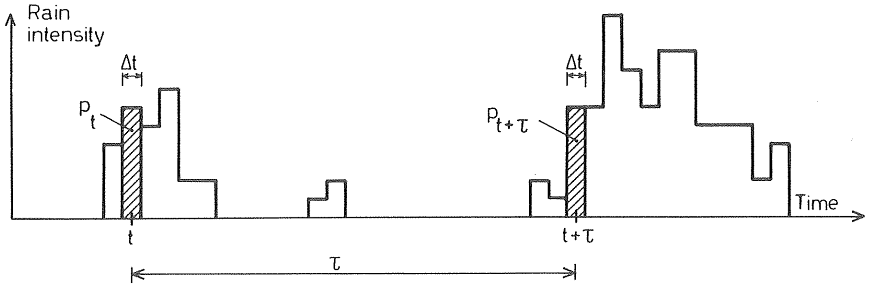


Fig. 1.1 Explanation of symbols used for autocorrelation analysis.

where p_t and $p_{t+\tau}$ = the rainfall depths at the time increments t and $t+\tau$, respectively.

r_τ = the correlation between p_t and $p_{t+\tau}$

The autocovariance, cov $(p_t, p_{t+\tau})$, and the variance, var p_t, were calculated by the equations

$$\text{cov}\ (p_t, p_{t+\tau}) = \frac{1}{N} \sum_{t=1}^{N-\tau} (p_t - \bar{p})(p_{t+\tau} - \bar{p}) \qquad \ldots(1.3)$$

$$\text{var}\ p_t = \frac{1}{N} \sum_{t=1}^{N} (p_t - \bar{p})^2 \qquad \ldots(1.4)$$

where N = total number of time intervals

$\bar{p}$ = mean value of p_t

The analysis was applied to data from each year for the period of June-November, which is the period when the heaviest storms occur. A time interval, Δt, of 10 minutes was used to reduce the computer costs. The influence of the length of the time interval is treated later in this section.

The result of the autocorrelation analysis is found in Table 1.1, and an example of the correlogram for one year is found in Fig. 1.2. The rainfall values are assumed to be independent after, on the average, 4 hours or longer, when the value of the correlation coefficient reaches its first low value, after which it may increase again. The correlation before the first minimum value is the correlation within one rainfall event. The existence of several minimum values might be explained by physical phenomena such as characteristic time intervals between different convective cells and frontal systems. The remaining correlation after about four hours might also be a result of a positive contribution from the zero rainfall values to the covariance. If no physical interpretation of the correlation coefficient is possible, the correlation coefficient may be assumed to be normally distributed and the

Table 1.1. Values of the shortest time-lag between independent successive 10-min rain-intensity values estimated by autocorrelation analysis. Values for the period of June-November 1921-1939, Lundby, Göteborg.

Year	Time h	Year	Time h	Year	Time h
1921	5.5	1928	6.0	1934	4.0
1923	4.5	1929	3.5	1935	3.0
1924	4.0	1930	4.0	1936	3.0
1925	2.5	1931	3.5	1937	4.0
1926	4.0	1932	3.0	1938	4.0
1927	4.0	1933	5.0	1939	4.0

Average value = 4.0 Standard deviation = 0.9

intensity values assumed to be independent when the correlation coefficient value is within a chosen confidence interval (see, for example, Yevjevich, 1972). For a 95% confidence limit value of 0.012, the correlation coefficient falls below this value after about 11 hours on the average. The use of 11 hours instead of 4 hours would have led to very long rainfalls with long rain-free periods within the events, during which, in many cases, the water in surface depression storages would evaporate. This will happen in fewer cases for the storms defined by the rain-free time period of 4 hours.

The significance of the length of the time interval, Δt, was investigated by replacing the 10-minute value used by 5 minutes and 60 minutes and calculating the correlograms for these intervals. The first minimum value (see Fig. 1.2) is reached after about the same time for the different time intervals even though the absolute values may vary.

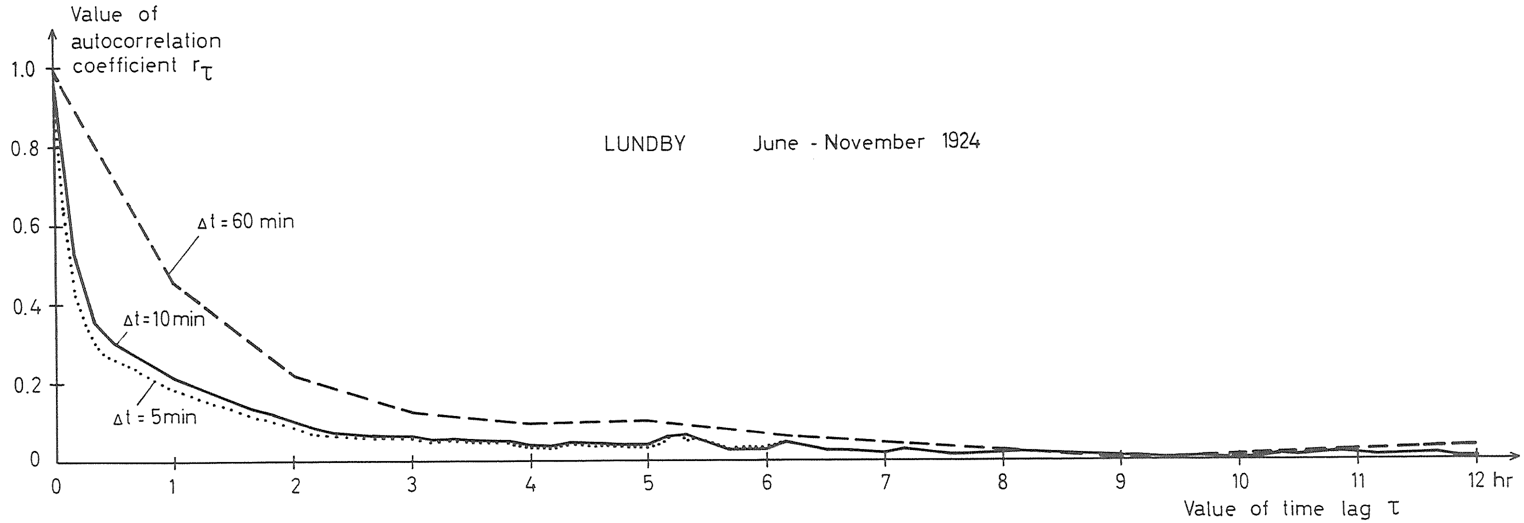


Fig. 1.2 *Correlogram showing the correlation between 5-, 10-, and 60-minute rain-intensity values at different time lags. Example for the period June-November 1924, Lundby, Göteborg.*

Statistical independent rainfall events are thus separated from each other by rain-free periods or periods with a rain intensity below 0.1 mm/h during 4 hours, or longer (see Section 5.3). Four hours can also be seen as a compromise between the wish to have long time intervals to include large runoff areas (long time of concentration) and be able to use the data for analysis of retention basins, and the wish to get a correct description of the surface hydrology (surface depression storage and evaporation).

Examples of results of similar studies with correlation techniques are 14 hours used by Howard (1976), 4.5 hours used by Wenzel and Voorhees (1978), and 2.3 hours used by Grace and Eagleson (1967). Without any special analysis, Marsalek (1978b) has used a time between rainfalls of 3 hours and Johansen (1979) 1 hour in studies of "design storms" similar to the study described in this report. When evaluating Intensity-Duration-Frequency curves, Arnell (1974) segregated storms by a rain-free inter-event time of 1 hour, DIF (1974) used 6 hours and Dahlström (1979) used 0.5 hour. Dahlström also states that there will be only a small change in the resulting Intensity-Duration-Frequency curves if the time between storms is changed within the range of 10 to 60 minutes.

Thus, there is a great variation in the lengths of rain-free periods between storms used in the studies. It is assumed that this is not a significant factor and that it has only a small influence on the result of this investigation.

2. INTENSITY-DURATION-FREQUENCY RELATIONSHIPS

2.1 Method of Development of Intensity-Duration-Frequency Relationships

Many of the design storms described in this report are in one way or another connected with the Intensity-Duration-Frequency curves (I-D-F curves). Therefore, the assumptions behind the curves and the method of development will be given here. I-D-F curves have also been evaluated for the Lundby rainfall station in Göteborg.

Intensity-Duration-Frequency curves can be evaluated on the basis of data from one or several rainfall stations. Each station is treated separately, and mean-value curves can be calculated. The rainfall series is first divided into separate statistical independent rainfall events. The criterion used is normally the time between the rainfall events and the assumptions behind are the same as those stated in Section 1.3. In most of the studies reported, the time interval is chosen arbitrarily and varies between 0.5 and 6 hours. In the present study 4 hours is used.

For a number of durations, the maximum average intensities are searched for each storm (see Fig. 2.1). For durations

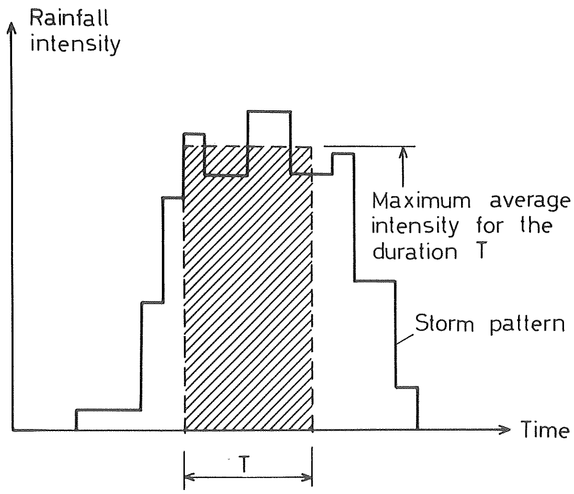


Fig. 2.1 Maximum average intensity for a given duration.

longer than the total real rainfall duration, the maximum average intensity is calculated as the total rainfall amount divided by the duration. For example, if a real rainfall lasts for 30 minutes and includes a total rainfall amount of 20 mm, the maximum intensity for a duration of 40 minutes would be (20/40) x 60 = 30 mm/h, and for a duration of 60 minutes (20/60) x 60 = 20 mm/h.

The maximum intensity values of the different durations for each rainfall event are then statistically prepared separately for each duration. The intensity values are ranked in descending order of size, and by means of a plotting formula, the statistical distribution function is estimated for each duration. A mathematical fit of a theoretical distribution function may be done or an eye-guided line may be drawn. Finally, values for the different durations for a specified recurrence interval are plotted with the duration as the abscissa and the maximum average rainfall intensity as the ordinate. The smoothed curve fitted to these points is called an Intensity-Duration-Frequency curve. Different curves are evaluated for different frequencies or return periods. Each I-D-F curve may contain data from different historical storms as each duration was treated separately, and this fact means that the return period for the complete curve is longer than for the individual points of the I-D-F curve.

As can be seen in Fig. 2.1, the maximum average intensity for a specified duration represents only a part of the total volume of the real rainfall. The rain volumes prior to and after the duration studied are not included. Especially the rainfall prior to the main rain burst should influence the design of retention basins. Fig. 2.2 shows the share of the total rain volume that on the average is included in the maximum-average-intensity part of the rainfall. This share varies with the duration from about 35% for a duration of 5 minutes to 75% for a duration of 240 minutes. Further details of the data behind Fig.2.2 are given in Section 2.2. The high percentage for longer durations is, among other things, an effect of the "ex-

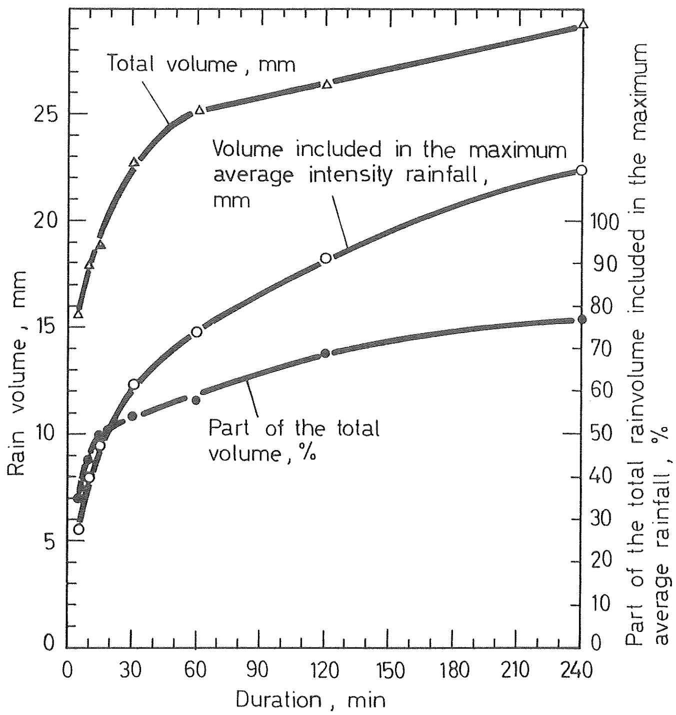


Fig. 2.2 *The part of the total rain volume that is included in the maximum average intensity for different durations. Average values for rainfalls with a return period exceeding 1/2 year. Data from Lundby, Göteborg, 1921-1939.*

tended durations" beyond the real total durations of the rainfalls. This means that for some rainfalls of longer durations, the total rain volume is included in the maximum average rainfall.

Furthermore, the I-D-F-curves give no information about the time variation of the rainfalls. This variation is of importance when designing and analyzing storm-water pipe systems, retention basins, and overflow constructions.

It should be pointed out that the I-D-F curves are evaluated for use in connection with the "rational method", and give in this case enough statistical information concerning the rainfall. Problems and errors arise when data from the I-D-F curves are used in other types of storm-water design models.

2.2 Intensity-Duration-Frequency Relationships for Lundby, Göteborg

Intensity-Duration-Frequency curves have been developed for Lundby, Göteborg. Data for the period of 1921-1939 have been treated, excluding the year 1922 because of periods of missing data for that year. General information about the rainfall station and the data is given in Chapter 5.

The total rainfall series was divided into separate rainfall events, where the time with no rainfall between the events was chosen to be 4 hours. This was the time required to obtain independent rainfall events, as determined in Section 1.3. Statistical information concerning the events is given in Chapter 5.

The maximum average intensities for durations from 5 minutes to 4 hours were evaluated for each rainfall. Observe that the method of extended durations was used, so for durations longer than the real rainfall duration, the maximum average intensities were calculated as the total rain volumes divided by the durations.

Since we were interested in extreme values only, the material was statistically treated as intensity values over a threshold. The aim was to include return periods of about 1/3-1/5 year. Therefore, the 90 largest intensity values for each duration were selected for the statistical analysis.

For each duration the maximum average intensity values were ranked in descending order and plotted on a statistical probability paper using the plotting formula

$$y_i = \sum_{j=1}^{i} \frac{1}{N+1-j} \; ; \; i = 1, 2, \ldots, N \qquad \ldots(2.1)$$

where y_i = plotting position for the evaluated intensity values in increasing order.

y_i = ln F; where F is the return period

N = number of treated intensity values which are chosen as equal to the number of treated time periods.

Eq. (2.1) estimates plotting positions for the exponential distribution according to the Natural Environment Research Council (1975). The plottings were made on an exponential distribution paper with a logarithmic scale for the intensity values. A few of the plotted distribution functions are shown in Fig. 2.3.

Different mathematical distribution functions were tested on the data, and the function log-Pearson Type III was found to be best and fitted to the plotted data by means of a computer program published by Kite (1977). In this program the parameter values can be estimated either by fitting the distribution directly to the data, or to the logarithms of the intensity values. The method of moments or the maximum likelihood method can be used. In this case the method of moments was used, applied to the logarithms of the intensity values. Some of the resulting distribution functions are shown in Fig. 2.3.

A χ^2-test was carried out to test the goodness of fit of the log-Pearson Type III distributions. Ten classes were used. The result of the test is shown in Table 2.1. At a significance level of 5%, the χ^2-value for a duration of 120 min is rejected. In the other cases the χ^2-values are below the values corresponding to the significance level of 5%. The log-Pearson Type III distribution is assumed to be applicable in this study, which is also confirmed by the agreement between the plotted intensity values and the fitted mathematical distributions shown in Fig. 2.3.

The resulting Intensity-Duration-Frequency values were estimated by the mathematical distribution functions, and values for the return periods 1/3, 1/2, 1, 2, 5, and 10 years are listed in Table 2.2. The I-D-F curves drawn as eye-guided lines between the values listed in Table 2.2 are shown in Fig. 2.4.

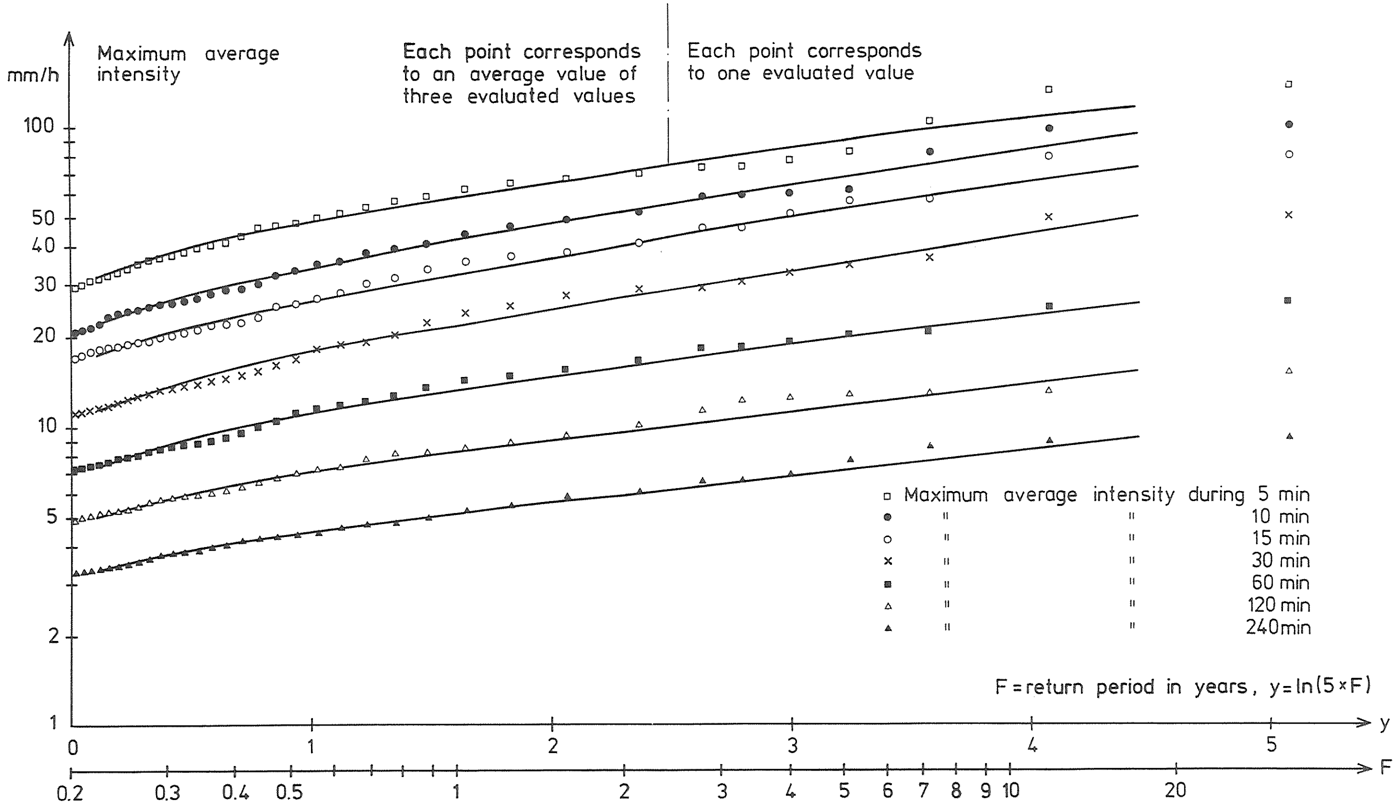


Fig. 2.3 Plotted distribution functions of maximum average intensity values with estimated mathematical functions of the type log-Pearson Type III. Lundby, Göteborg, 1921–1939.

Table 2.1. Result of χ^2-test of the fitted log-Pearson Type III distributions to evaluated maximum average intensity values for Lundby, Göteborg, 1921-1939.

Duration min	χ^2	$P(\chi^2)$
5	4.0	<0.35
10	6.9	<0.70
15	12.0	<0.95
30	11.6	<0.95
60	10.9	<0.90
120	17.6	~0.99
240	6.4	<0.60

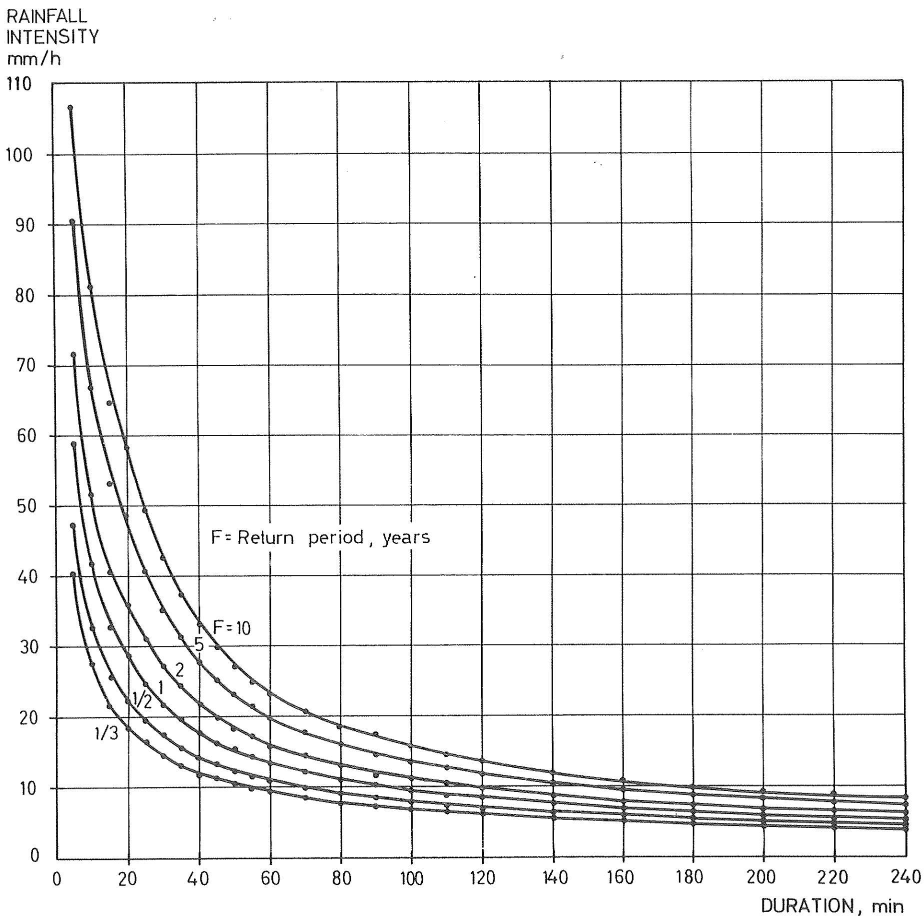


Fig. 2.4 Intensity-Duration-Frequency curves for Lundby, Göteborg, 1921-1939.

Table 2.2. Values of maximum average intensities (mm/h) for different return periods and different durations estimated by the log-Pearson Type III distribution function. Lundby, Göteborg, 1921-1939.

Duration	Return period, year					
min	1/3	1/2	1	2	5	10
5	40.3	47.3	59.0	71.5	90.4	106.8
10	27.9	32.9	41.6	51.5	67.0	81.0
15	21.9	25.8	32.8	40.7	53.2	64.8
20	18.7	22.3	28.7	36.0	47.6	58.3
25	16.4	19.5	24.9	31.1	40.7	49.5
30	14.5	17.2	21.9	27.1	35.2	42.5
35	13.1	15.5	19.7	24.2	31.1	37.3
40	12.0	14.1	17.8	21.7	27.8	33.2
45	11.1	13.0	16.3	19.9	25.2	29.9
50	10.4	12.2	15.2	18.3	23.1	27.2
55	9.9	11.5	14.2	17.1	21.3	25.0
60	9.3	10.9	13.4	16.0	19.8	23.1
70	8.5	9.8	12.1	14.3	17.7	20.5
80	7.8	9.0	11.0	13.0	16.0	18.5
90	7.2	8.3	10.1	12.0	14.7	17.0
100	6.8	7.7	9.4	11.0	13.5	15.6
110	6.4	7.3	8.8	10.4	12.6	14.5
120	6.1	6.9	8.3	9.8	11.9	13.6
140	5.5	6.3	7.4	8.7	10.5	12.0
160	5.1	5.7	6.7	7.8	9.4	10.7
180	4.7	5.3	6.2	7.2	8.6	9.8
200	4.4	4.9	5.8	6.7	8.0	9.2
220	4.1	4.6	5.4	6.3	7.6	8.7
240	3.9	4.3	5.1	5.9	7.1	8.2

To each curve was fitted a mathematical expression of the form

$$i_m = \frac{a}{T+b} + c \qquad \ldots(2.2)$$

where i_m = maximum average intensity during the time T

T = duration

a, b, c = constants

The constants a, b, and c were determined by the method of least squares, and the results are given in Table 2.3, together with the standard errors of the estimates given by Eq. (2.2). The Eq. (2.2) was found to better fit the data than an expression of the form $i_m = a/(T+b)^c$.

Table 2.3. Values of the constants a, b, and c in the intensity formula $i_m = a/(T+b) + c$, and standard errors of the estimates made by the formula. i_m is obtained in mm/h and T is given in minutes, $5 \text{ min} \leq T \leq 240$ min. Lundby, Göteborg, 1921-1939.

Return period year	Constants a	b	c	Standard error mm/h
1/3	445	7	2.5	0.34
1/2	535	7	2.5	0.41
1	725	8	2.5	0.45
2	965	9	2.0	0.51
5	1325	10	1.5	0.66
10	1700	11	0.5	1.02

3. DESIGN STORMS

3.1 Definition of Design Storms

The different types of synthetic rainfalls are in this report called Design Storms to separate them from the real unchanged historical storms.

> A Design Storm is a rainfall which is developed for design of specified types of objects, for instance pipes or retention basins. A Design Storm is coupled with a return interval, and the flow value which is calculated by means of the storm, is supposed to have the same return interval as the storm (see also Patry and McPherson, 1979).

The design storms may be divided into different types of storms for the design of different parts of a storm-sewer system. One storm can be developed for the design of pipe sizes for carrying peak flows, while another storm can be developed for the design of sizes of basins for retention of runoff volumes. The former is to at least give correct peak flows, while the latter is to give correct runoff volumes, including the hydrographs. For the estimation of overflow volumes for longer time periods, another type of design storm is needed. Most of the design storms used today are storms evaluated for the calculation of peak flows, but they are also used for the design of retention basins, which may not be correct. Examples of these storms are the Chicago storm (Keifer and Chu, 1957), the Sifalda rainfall (Sifalda, 1973), and the English rainfall (Natural Environment Research Council, 1975). A special type of simplified storms for the analysis of overflow volumes was used by Lindholm (1974, 1975).

The design storms are related to specific return intervals, for example, a one-year storm. One or several parts of the rainfalls are related to rainfall statistics. In most cases they are related to the Intensity-Duration-Frequency curves. For example, the whole Chicago storm follows an I-D-F curve, while for the Sifalda rainfall, only the central part is related to the I-D-F curves, and the rainfall prior to and after the central part is just an average rainfall for many heavy storms.

In the following different design storms are described. First a general description of the storm is given, mainly taken from the literature, after which follows comments on advantages and disadvantages of the characteristics of the storm. Finally, local coefficients for the storm were estimated from the data for Lundby, Göteborg, 1921-1939.

The local coefficients were estimated for the heaviest historical rainfalls, only. These rainfalls were identified from the ranking lists of the maximum average intensities for different durations described in Section 2.2. All rainfalls with maximum average intensity values, for one or several selected durations, above a selected threshold (return period) were chosen for the estimation of the coefficients. This method was chosen because all design storms are in one way or another connected with the Intensity-Duration-Frequency relationships.

3.2 Average-Intensity-Duration (I-D-F) Design Storm

General Description

To use the maximum average intensity for a specified duration obtained from the Intensity-Duration-Frequency curves is the simplest form of a design storm. The storm is evaluated for the design frequency from the I-D-F curves, as shown in Fig. 3.1. The rainfall is fed into the design models as a constant intensity during the chosen duration.

When a system is designed, rainfalls with different durations are tested to find the durations that gives the maximum peak flows or runoff volumes. The upstream parts of a sewer system are designed for short rainfall durations, while the downstream pipes are sized for longer durations. Since the pipe sizes are unknown in the design

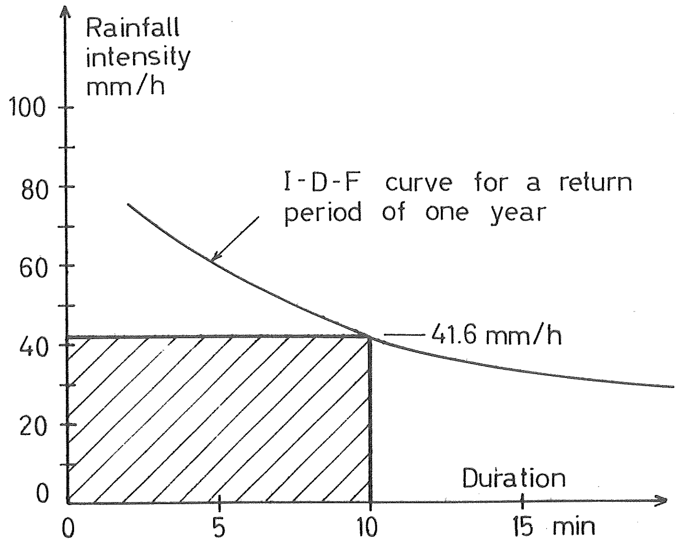


Fig. 3.1 *Example of the evaluation of an Average-Intencity-Duration design storm from an Intensity-Duration-Frequency curve. A constant intensity of 41.6 mm/h during 10 minutes. Return period one year.*

case and since the sizes of the upstream pipes influence the propagation of the hydrograph, the design should start in the upstream end with short rainfall durations and then proceed downstream with increasing durations.

Characteristics of the Storm

The Average-Intensity-Duration storm has been used in the time-area method and originates from the use of the rational method. There is a high correlation between the peak-flow values and the maximum average intensity values if the duration of the storm is estimated correctly.

One disadvantage with this simplified design storm is that it represents only a part of the total rain volume of the real rainfalls, as can be seen in Fig. 2.2. Another disadvantage is that the variation in rain intensity during the rainfall is not described. These two drawbacks should be of importance, at least, for the design and analysis of retention basins and overflows.

Local I-D-F Design Storm

For the comparative designs of the storm sewer systems described in Section 6.3, values obtained from the Intensity-Duration-Frequency curves for Lundby, Göteborg, were used (see Table 2.2).

3.3 Chicago Design Storm

General Description

In 1957 Keifer and Chu presented a design storm developed from the total Intensity-Duration-Frequency curves. The most important characteristic of this storm is that the maximum average intensities for all durations of the storm follow an I-D-F curve (see Fig. 3.2). The easiest way of developing such a rainfall is to assume that the peak intensity is located in the middle of the rain and distribute the rain symmetrically around the peak. That type of storm has been used by Lindholm (1974) and Thorndal (1971).

The Chicago design storm was developed from the mathematical expressions for the I-D-F curves, and Keifer and Chu (1957) found the following three rainfall characteristics to be the most important ones affecting the peak runoff rate.

1. The volume of water falling within the maximum period.
2. Amount of antecedent rainfall.
3. Location of the peak rainfall intensity.

The I-D-F curves for Lundby, Göteborg, were approximated by the following type of mathematical expression,

$$i_m = \frac{a}{T+b} + c \qquad \ldots(3.1)$$

where i_m = maximum average intensity during the time T

T = duration

a,b,c = constants

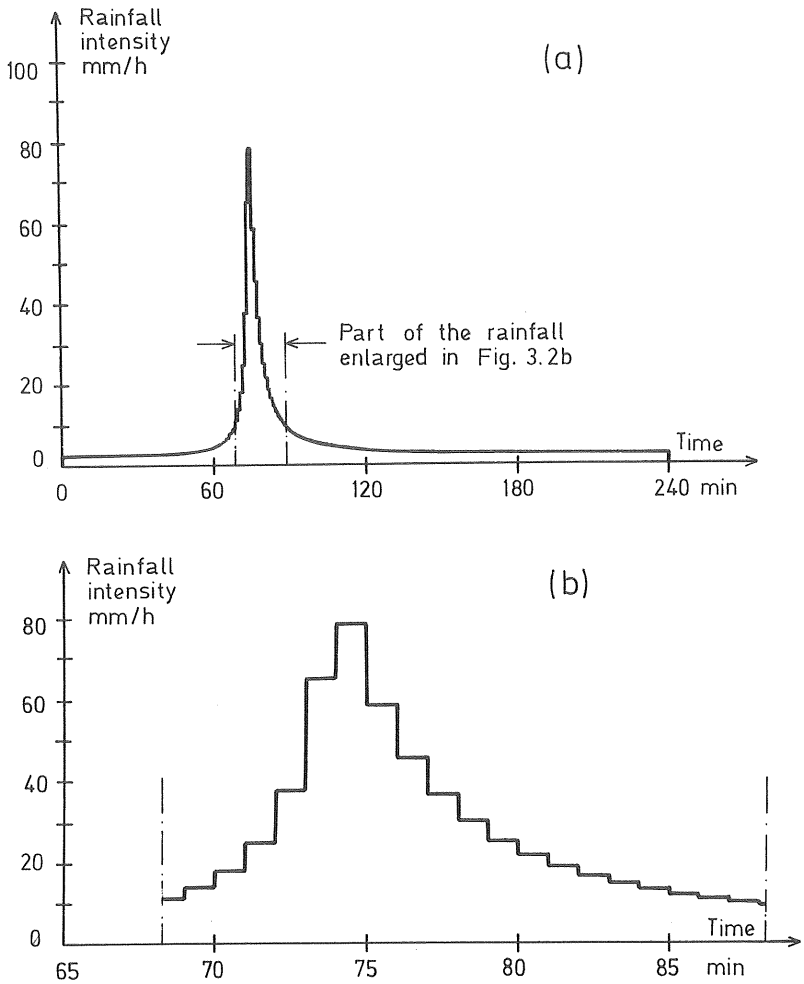


Fig. 3.2 *Design rainfall, suggested by Keifer and Chu (1957), derived from the Intensity-Duration-Frequency relationship for Lundby, Göteborg, 1921-1939. Recurrence interval one year.*
Fig. (a) shows the complete storm, and
Fig. (b) shows the central part enlarged.

The rain volume falling during the time T is

$$P = \left(\frac{a}{T+b} + c\right) \cdot T \qquad \text{...(3.2)}$$

where P = rain volume

A rainfall hyetograph (hyetograph = time-variation of rainfall intensity) can now be developed which has the peak located at the beginning of the rainfall and, with no antecedent rainfall, a so-called completely advanced rainfall pattern (see Fig. 3.3). The volume of rainfall included in the hyetograph is:

$$P = \int_{0}^{T} i \, dt \qquad \text{...(3.3)}$$

where $\quad i$ = rain intensity

Differentiating the Equations (3.2) and 3.3) and setting them equal gives:

$$i = \frac{a \cdot b}{(t+b)^2} + c \qquad \text{...(3.4)}$$

The hyetograph described by Equation (3.4) has the same maximum average intensity for any duration as that given by the corresponding I-D-F curve. An intermediate rainfall pattern can now be developed if the duration T is

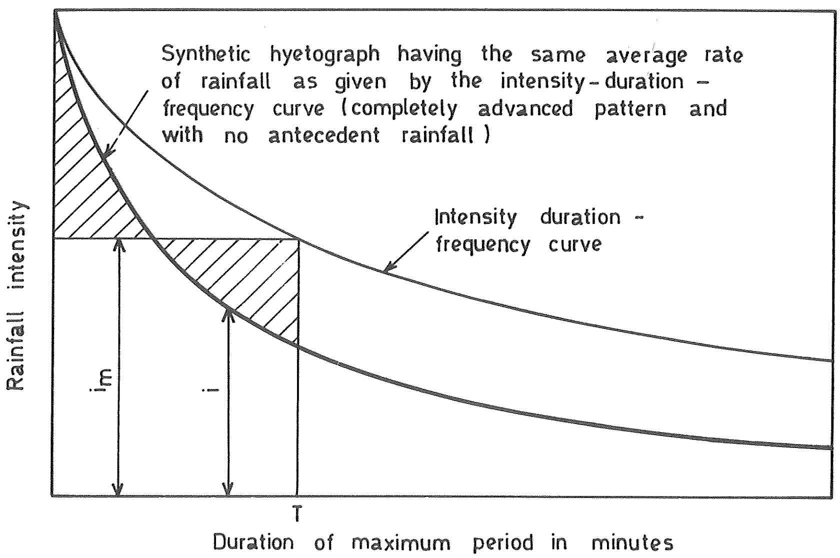


Fig. 3.3 A synthetic rainfall hyetograph having a completely advanced pattern and with no antecedent rainfall developed from the Intensity-Duration-Frequency curve. After Keifer and Chu (1957).

split into two parts, where $T_f = r \cdot T$ is the part which occurs before the peak and $T_e = (1-r)T$ is the part occurring after the peak. The sum of T_f and T_e is equal to T, and the completely advanced rainfall pattern given by Equation (3.4) occurs for $r = 0$. If $t = t_f/r$ and $t = t_e/(1-r)$ are inserted in Equation (3.4), we obtain two equations describing the rain-intensity pattern prior to the peak and after the peak (see also Fig. 3.2),

$$i = \frac{a \cdot b}{\left(\frac{t_f}{r} + b\right)^2} + c \quad \text{(before the peak)} \qquad \ldots(3.5)$$

$$i = \frac{a \cdot b}{\left(\frac{t_e}{1-r} + b\right)^2} + c \quad \text{(after the peak)} \qquad \ldots(3.6)$$

where t_f = time counted from peak intensity towards the start of rainfall

t_e = time counted from peak intensity towards the end of rainfall

r = relationship between the time prior to peak intensity (T_f) and the total duration (T) $r = T_f/T$; $1-r = T_e/T$

The Equations (3.5) and (3.6) give a rainfall which has the same maximum average intensities (and rainfall volumes) for any duration as that given by the corresponding I-D-F curve.

The location of the peak intensity, or the value or r, is found in one of two ways. One is to study the location of the peak intensity within the duration T, and the other way is to evaluate the antecedent rain volume before the period T having the maximum average intensity. Keifer and Chu (1957) used both methods and got about the same result with both of them. They evaluated r to be about 15/40 for some places in Chicago. Preul and Papadakis (1973) found r to be 13/40 for Cincinnati, Ohio, and Bandyopadhyay (1972) got r = 16/40 for Gauhati, India. Sifalda (1973), who developed another type of design storm(see Section

3.4), evaluated r for three places in Czechoslovakia and found the value to be approximately 14/40.

Characteristics of the Storm

The Chicago Design Storm described above now meets the assumptions concerning important rainfall characteristics stated by Keifer and Chu (1957). Besides, it is easy to evaluate and is valid for all durations. If the duration T is chosen long enough, it will probably include on the average most of the total rain volume, since antecedent rainfall and rainfall after the maximum period T are decreasing as T increases (see Fig. 2.2).

The Chicago Design Storm was developed to agree with a complete I-D-F curve. Since the different durations are treated separately, however, when an I-D-F curve is developed, one curve may consist of data from several historical rainfalls. This should give the Chicago Design Storm a return period which is longer than the return period for the individual points of the corresponding I-D-F curve. Moreover, it has an unnaturally high peak intensity, which can be slightly smoothed when transformed to input data for a runoff model. The choice of the total duration T for different runoff simulations seems to be a problem, and no rules are available. Keifer and Chu (1957) chose the longest estimated time of concentration for Chicago as the value of T.

Local Chicago Design Storm

For the comparative simulations of the runoff for different design storms and historical storms, a design storm of the Chicago-type was evaluated from the data for Lundby, Göteborg for 1921-1939.

The rain-intensity pattern is described by Eqs. (3.5) and (3.6), where the values of the constants a, b, and c are obtained from Table 2.3. The value of the relationship, r, between the time prior to peak intensity and the total duration was estimated by studying the location in time of the heaviest part of the storm for a number of historical rainfalls.

The total duration of the Chicago-type design storm was determined to be 240 minutes, which is the longest duration for which the I-D-F curves (see Fig. 2.4) are valid.

In the ranking lists of maximum average intensities for the durations from 5 minutes to 240 minutes, the historical rainfalls were identified for which at least one of the maximum average intensities for any of the durations is larger than the intensity value corresponding to a specified return period. The total number of storms obtained for each return period is listed in Table 3.1.

The value of the constant r was estimated as the time from the beginning of the 240 minutes maximum average intensity rainfall to the middle of the duration of the different maximum intensity rainfalls divided by 240 minutes. This was done for each rainfall, and mean values were calculated for each duration and each return period (see Table 3.1). For historical rainfalls with a total duration less than 240 minutes, the total duration was used instead of 240 minutes.

Table 3.1 Results of estimations of the relationship, r, between the time prior to peak intensity and the total duration of 240 minutes of the Chicago-type design storm. Lundby, Göteborg, 1921-1939.

F	Total number	r for different durations (min)					
year	of storms	5	10	15	30	60	120
1/5	165	0.41	0.41	0.42	0.43	0.46	0.48
1/3	109	0.42	0.41	0.41	0.43	0.45	0.47
1/2	72	0.41	0.40	0.40	0.42	0.45	0.47
1	41	0.32	0.34	0.33	0.34	0.39	0.42
2	22	0.33	0.32	0.30	0.31	0.35	0.43
5	13	0.34	0.32	0.31	0.32	0.37	0.45
10	7	0.34	0.31	0.30	0.30	0.36	0.47

In the Chicago design storm, a constant value of r is used for all durations. Therefore, average values were calculated; one value was calculated for return periods shorter than one year and one value for return periods of one year and longer. This division into two values is appropriate, since there seems to be a clear difference in the values in Table 3.1 for return periods shorter and longer than one year. The resulting r-values are shown in Table 3.2.

Table 3.2 *Average values of the relationship, r, between the time prior to peak intensity and the total duration of 240 minutes. Lundby, Göteborg, 1921-1939.*

F year	r
$1/5 \leq F \leq 1/2$	0.43
$1 \leq F \leq 10$	0.35

The values given in Table 3.2 agree with the values found in earlier investigations and mentioned in this Section. An example of an estimated design storm of the Chicago type is shown in Fig. 3.2.

3.4 Sifalda Design Storm

General Description

The design storm presented by Sifalda (1973) is compounded of the maximum average intensity duration storm obtained from the I-D-F curves with precipitation added before and after the maximum duration (see Fig. 3.4). The rainfall was estimated as an average rainfall for those historical rains, where the maximum average intensity for at least one duration from 5 minutes to 120 minutes exceeded the I-D-F curve with a recurrence interval of one year.

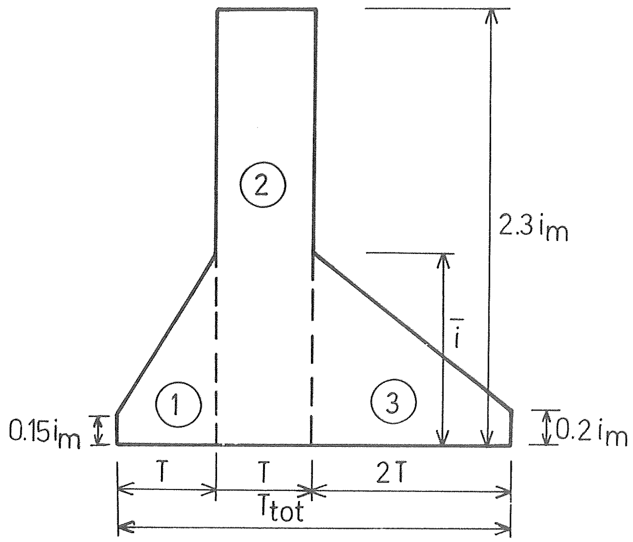


Fig. 3.4 *Design rainfall suggested by Sifalda (1973). Intensity-duration for part ② is obtained from Intensity-Duration-Frequency curves.*

Sifalda (1973) evaluated rainfall registrations for three places in Czechoslovakia. In all, 91 rainfalls were included in the evaluation. The intensities were calculated with successive time steps of 2.5 minutes. Precipitation at the beginning of the rainfall was rejected if the rain intensities were below 0.1 mm/min and at the same time the volume of this rain was less than 0.5 mm. At the end of the rainfall, precipitation was rejected if the rain intensities were below 0.1 mm/min and at the same time the volume was less than 0.5 mm during the next five minutes. Successive rainfalls were treated as separate rain events if the period with no rainfall exceeded the duration of the first rainfall. Average values of some rainfall characteristics are listed in Table 3.3 (see also Fig. 3.5, where some parameters are defined).

Characteristics of the Storm

By means of the characteristic rainfall parameter values listed in Table 3.3, Sifalda (1973) suggested the design storm shown in Fig. 3.4. Of the total rain volume, the main part ② contains 56%, the part ① before the main

Table 3.3 Rainfall characteristics according to Sifalda (1973).

Station	P_{tot} mm	T_{tot} min	$\frac{P_{tot}}{T_{tot}}$ mm/min	k_i -	D min	T_B min	T_S min	d %	t_B %	t_S %	P_S mm	p_S %	F year
Prag (29 Rainfalls)	18.0	32.2	0.56	2.2	6.7	8.0	11.2	32.1	20.3	35.7	6.65	37.5	3.3
Pilsen (20 Rainfalls)	20.4	34.3	0.60	2.4	6.5	7.5	10.4	21.1	23.6	33.7	8.05	38.8	6.1
Brünn (42 Rainfalls)	22.2	34.3	0.65	2.3	7.4	7.9	11.2	27.3	23.4	34.6	10.9	40.2	7.5

P_{tot} = total rain volume during T_{tot}

T_{tot} = total duration

$\bar{i}$ = P_{tot}/T_{tot}

i_{max} = maximum intensity

k_i = $i_{max}/\bar{i}$

D = duration of period with maximum intensity

T_B = time to beginning of period with maximum intensity

T_S = time to peak intensity

d = $100 \cdot D/T_{tot}$

t_B = $100 \cdot T_B/T_{tot}$

t_S = $100 \cdot T_S/T_{tot}$

P_S = rain volume before peak intensity at time T_s

p_S = $100 \cdot P_S/P_{tot}$

F = average return period

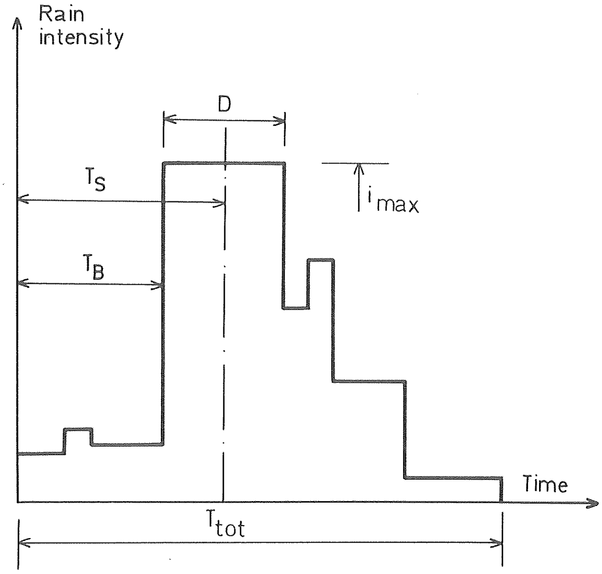


Fig. 3.5 *Definition of some rainfall parameters evaluated by Sifalda (1973). Average parameter values listed in Table 3.3.*

part 14%, and the part 3 after the main part 30%. The value, 56%, can be compared with data from the measurements at Lundby, Göteborg, shown in Fig. 2.2. As can be seen in Fig. 2.2, the part of the total rain volume included in the maximum average intensity part increases as the total duration increases. The Sifalda design storm should therefore be different for different durations with the variations in total volumes expressed by different volumes of the part 1 before and part 3 after the main part 2. The storms should also be different for different return periods depending on the great variation in the intensity of part 2 for different return periods.

Most of the disadvantages with the use of the maximum-average-intensity-duration design storm are included when using the Sifalda design storm. The volumes are better described, but the rainfall gives no information concerning the time variation of the rain intensities for the heaviest parts of the historical storms.

Local Sifalda Design Storm

Parameters for a Sifalda-type design storm were estimated from the data for Lundby, Göteborg, 1921-1939, to make possible a comparison of the result of runoff simulations with a Sifalda-type storm with the result of runoff simulations with historical storms.

The evaluation of the parameter values was done for each duration and different return periods separately. This was done to avoid some of the disadvantages described above. In the ranking list of maximum average intensities for each duration, the historical rainfalls were identified for which the maximum average intensity value exceeded the value corresponding to a specified return period. For those rainfalls, average values of total duration, durations of the parts prior to and after the maximum average intensity, and volumes of the different parts of the rainfall were evaluated. The result is presented in Table 3.4.

The parameter values in Table 3.4 together with values of the maximum average intensities obtained from Table 2.2 were used to construct design storms. Linear regression analyses were carried out between the different parameter values and the duration of the maximum average part of the rainfall. This was done for the return periods 1/3, 1/2, 1, 2, and 5 years. The regression relationships were then used for the estimation of the Sifalda storms. The result of the regression analysis and the steps in the estimation of the storms are described in Appendix I.

The regression analysis of durations and volumes of part 3 gave higher values of the linear correlation coefficients than similar analysis of part 1 . Therefore, part 3 was used when the local Sifalda design storm was estimated. This method gave durations of the different parts (see Table 3.5) comparable with the durations evaluated from the historical storms and listed in Table 3.4. For the total volumes it was assumed to be important to

Table 3.4 *Evaluated values of parameters describing a Sifalda-type design storm. Average values for historical storms for Lundby, Göteborg, 1921-1939.*

Return period	Number of rain-falls	Duration of maximum average inten-sity	Duration of rainfall				Volume of rainfall			
			Total dura-tion	Part ①	Part ②	Part ③	Total volume	Part ①	Part ②	Part ③
year		min	min	%	%	%	mm	%	%	%
1/3	54	5	369	42	1	57	15.8	31	31	38
		10	403	41	3	56	17.2	22	40	38
		15	434	44	3	53	19.0	23	43	34
		30	464	41	6	53	21.4	20	50	30
		60	428	33	14	53	21.8	16	59	24
		120	495	36	24	40	24.4	16	67	17
		240	589	30	40	30	27.1	15	75	11
1/2	36	5	310	35	2	63	15.6	26	35	38
		10	409	41	3	56	17.9	22	44	33
		15	401	40	4	56	18.8	18	50	32
		30	400	34	8	58	22.7	15	54	30
		60	446	38	14	48	25.2	16	58	25
		120	481	31	25	44	26.4	14	69	17
		240	577	32	41	27	29.2	14	77	9
1	18	5	288	15	2	83	16.8	28	39	33
		10	235	26	4	70	16.5	16	59	25
		15	355	23	4	73	21.7	15	53	32
		30	349	21	9	70	22.6	9	67	24
		60	400	24	15	61	27.3	11	64	26
		120	488	35	25	40	32.7	15	66	19
		240	544	34	43	23	33.6	13	78	9
2	9	5	245	30	2	68	19.4	35	39	26
		10	241	19	4	77	18.2	15	65	20
		15	190	16	8	76	20.2	9	69	22
		30	313	28	10	62	27.2	6	66	28
		60	509	16	12	72	30.4	7	67	26
		120	399	32	30	38	35.0	11	72	17
		240	405	20	59	21	35.0	7	84	9
5	6	5	270	38	2	60	21.0	36	42	22
		10	231	21	4	75	20.9	18	64	18
		15	196	22	8	70	21.4	10	75	15
		30	254	24	12	64	25.4	4	88	8
		60	412	35	15	50	35.7	10	64	26
		120	357	29	34	37	33.9	8	38	14
		240	528	28	45	27	40.7	9	81	10

Table 3.5. Data concerning by regression analysis estimated design storms of the Sifalda-type.

Return period	Duration of maximum average intensity	Duration of rainfall				Volume of rainfall				
		Total duration	Part ①	Part ②	Part ③	Total volume	Part ①	Part ②	Part ③	Volume of part ②
year	min	min	%	%	%	mm	%	%	%	
1/3	5	367	43	1	56	15.6	43	22	35	3.36
	10	399	41	3	56	17.6	40	26	34	4.65
	15	418	41	4	55	18.8	37	29	34	5.48
	30	451	40	7	53	20.7	33	35	32	7.25
	60	483	38	12	50	22.7	31	41	28	9.30
	120	516	34	23	43	24.7	30	49	21	12.20
	240	548	27	44	29	26.6	33	59	8	15.60
1/2	5	328	39	2	59	15.6	41	25	34	3.94
	10	366	39	3	58	18.0	37	30	33	5.48
	15	389	38	4	58	19.5	35	33	32	6.45
	30	427	37	7	56	22.0	30	39	31	8.60
	60	466	36	13	51	24.4	28	45	27	10.90
	120	505	33	24	43	26.9	29	51	20	13.80
	240	543	29	44	27	29.4	34	59	7	17.20
1	5	243	23	2	75	15.3	38	32	30	4.92
	10	294	23	3	74	18.7	34	37	29	6.93
	15	323	22	5	73	20.7	31	40	29	8.20
	30	374	23	8	69	24.1	28	45	27	10.95
	60	425	24	14	62	27.5	26	49	25	13.40
	120	475	27	25	48	30.9	27	54	19	16.60
	240	526	35	46	19	34.3	32	59	9	20.40
2	5	212	24	2	74	17.0	40	35	25	5.96
	10	255	23	4	73	20.5	33	42	25	8.58
	15	280	23	5	72	22.6	30	45	25	10.18
	30	324	23	9	68	26.1	24	52	24	13.55
	60	367	23	16	61	29.6	24	54	22	16.00
	120	410	24	29	47	33.1	23	59	18	19.60
	240	454	28	53	19	36.6	26	64	10	23.60
5	5	188	30	3	67	18.0	40	42	18	7.53
	10	237	30	4	66	21.8	31	51	18	11.17
	15	266	29	6	65	24.1	27	55	18	13.30
	30	315	28	10	62	28.0	20	63	17	17.60
	60	364	28	16	56	31.8	22	62	16	19.80
	120	414	26	29	45	35.7	18	67	15	23.80
	240	463	26	52	22	39.6	17	72	11	28.40

obtain values close to the volumes evaluated from the historical storms. The volumes of part ③ were estimated by the regression equation (Eq. I.5) which gave the percentages of volumes included in part ③ . The volumes of part ② were taken from I-D-F relationships for Lundby, Göteborg, and the volumes of part ① were calculated as the differences between the total volumes and the volumes of part ② and ③ . The volumes of part ① , thus, became larger than the volumes evaluated from the historical storms because the volumes of part ② taken from the I-D-F curves were smaller than the volumes of part ② evaluated from the historical storms. The differences in the volumes of part ② depend on the fact that the volumes of part ② for the historical storms were evaluated for all historical storms above the return period under consideration. The larger volumes of part ① probably in-

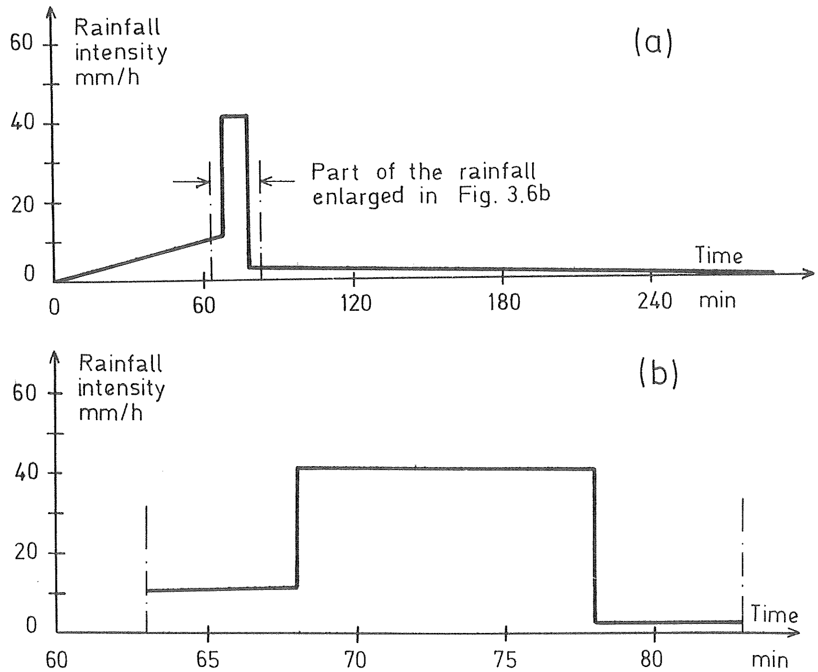


Fig. 3.6 Example of a design storm of the Sifalda-type, estimated from data for Lundby, Göteborg, 1921-1939. Return period equal to one year.
Fig. (a) shows the complete storm, and
Fig. (b) shows the central part enlarged.

crease the simulated peak-flow values in the pipe system, but as can be seen in Section 7.3, the Sifalda design storm, in spite of this, gave underestimated peak flows. The overestimation of the volumes of part (1) is, thus, assumed to be of minor importance when the storm is used for the simulation of peak-flow values. For the calculation of volumes of retention basins, one should check if it is more appropriate to use a regression equation of part (1), which comes prior to the main part (2) of the storm, instead of using a regression equation of part (3).

An example of a design rainfall of the Sifalda-type is shown in Fig. 3.6 and data concerning the estimated storms are given in Table 3.5.

3.5 Illinois State Water Survey (ISWS) Design Storm

General Description

A design storm developed at the Illinois State Water Survey was presented by Huff (1967). The total rain volume is obtained from the Intensity-Duration-Frequency curves, and the temporal rainfall distributions (see Figs. 3.7 and 3.8) are the average distributions for a number of rainfalls with a specified probability of occurrence connected to each curve, as will be explained later.

In the evaluation, a storm was defined as a rain period separated from the preceding and succeeding rainfalls by a rainfree period of at least 6 hours. Rain data from an areal network of instruments were used, and only rainfalls with network mean rainfall volumes exceeding 12.7 mm (1/2 inch) and/or the volumes for one or more gages exceeding 25.4 mm (1 inch) were utilized in the evaluation of the time distributions. The time resolution in the data was 30 minutes, and thus, data on showers of short durations were smoothed. The storms were also classified into four groups depending on whether the heaviest rainfall occurred in the first, second, third, or fourth quarter of the total storm period. The final time distributions

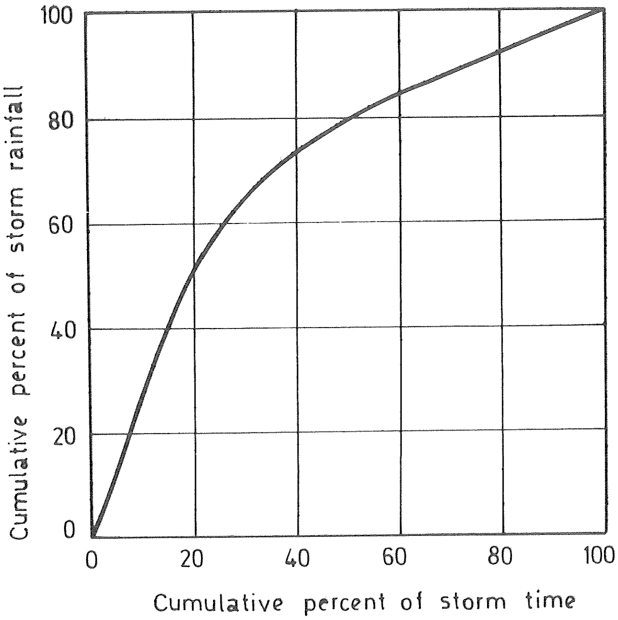


Fig. 3.7 *Example of the temporal rainfall distribution presented by Huff (1967) and suggested for use in the ILLUDAS-model according to Terstriep and Stall (1974).*

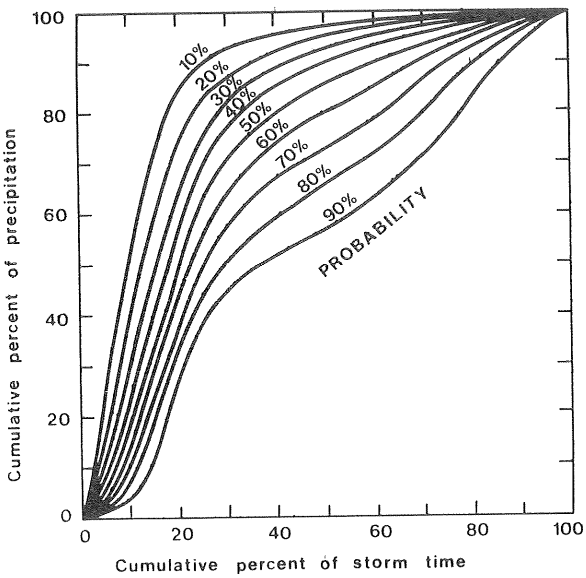


Fig. 3.8 *Time distributions of first-quartile storms according to Huff (1967).*

are average distributions for the rainfall in each quartile and are average curves for areas of 50 to 400 square miles. The curves are expressed in probability terms for each quartile. An example of the first quartile storms is shown in Fig. 3.8. If the over-all probability of a special time distribution is desired, for example, the 10% curve of the first-quartile storms, the probability of 10% is multiplied by the probability of obtaining a first-quartile storm, which is 30%. So the over-all probability of the 10% curve is 0.10 x 0.30 = 0.03 or 3%. Also presented by Huff (1967) are tabulated differences between the presented curve, Fig. 3.7, and curves for specific sizes of areas. The rainfall distribution shown in Fig. 3.7, which was suggested for use in the ILLUDAS runoff model by Terstriep and Stall (1974), is the first-quartile storm, 50% probability level, and is valid for point rainfall.

When using the Illinois State Water Survey Storm, one estimates the duration of the rainfall, and then the total volume of rainfall is obtained from Intensity-Duration-Frequency curves. The volume is then distributed in time according to the temporal rainfall distribution curve. The proper duration to use is the one which causes the greatest peak flow.

Characteristics of the Storm

The Illinois State Water Survey Storm was originally evaluated from data with a time step of 30 minutes. Later on, time steps of 15 minutes and 5 minutes have been used (see Vogel and Huff, 1977). For urban areas, at least in Sweden, these time steps are longer than the characteristic time step that must be used to accurately simulate urban runoff. A more suitable time step would be 1-5 minutes.

The overall probability can be obtained for the rainfall distribution curves, but we do not know if the same probability is obtained for the simulated peak flows, especially since the rainfalls were separated into different quartiles.

Local ISWS Design Storm

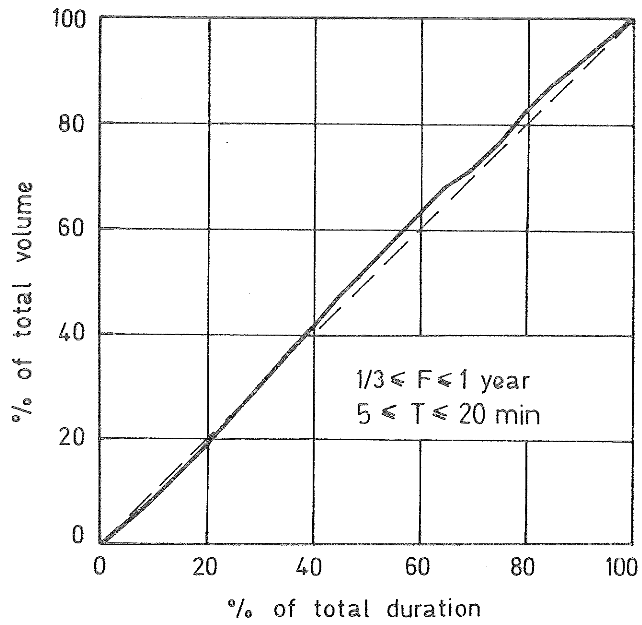
A design storm of the same type as the Illinois State Water Survey Storm was estimated from the data for Lundby, Göteborg. For each duration of the maximum average intensity periods the historical rainfalls were identified for which the maximum average intensity value was larger than the value corresponding to a specified return period. For all identified historical rainfalls, temporal rainfall patterns were then evaluated for each return period and within each duration of 5, 10, 15, 30, 60, 120, and 240 minutes. Average temporal patterns were calculated for each return period and for each duration.

No classifications, similar to those made by Huff (1967), were made of the rainfalls as to in which part of the storm the heaviest rainfall occurs. This was not done because it is difficult to judge the overall probability of the simulated peak flows.

The temporal rainfall pattern was evaluated within the maximum average intensity period only, because the total volume for the total duration is obtained from the I-D-F curves when the ISWS storm is used. The result is expressed as the cumulative percentage of precipitation as a function of the cumulative percentage of storm time, and is presented in Appendix II in cumulative time steps of 10% (the evaluation was made with a time step of 1%).

The different temporal rainfall patterns shown in Appendix II were grouped into four categories according to the shape of the curves. An average curve was calculated for each group and the result is shown in Fig. 3.9. The curves for the duration of 240 minutes were excluded from the average curves because they differed too much from the other curves.

Rainfall hyetographs for the runoff simulations were then estimated for different durations by taking the total volumes from the I-D-F curves and distributing the volumes according to the cumulative values obtained from Fig. 3.9. An example of a hyetograph is shown in Fig. 3.10.



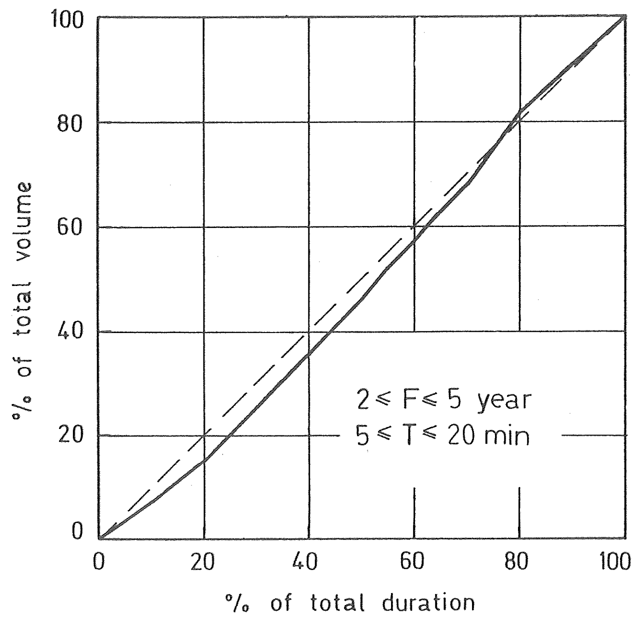
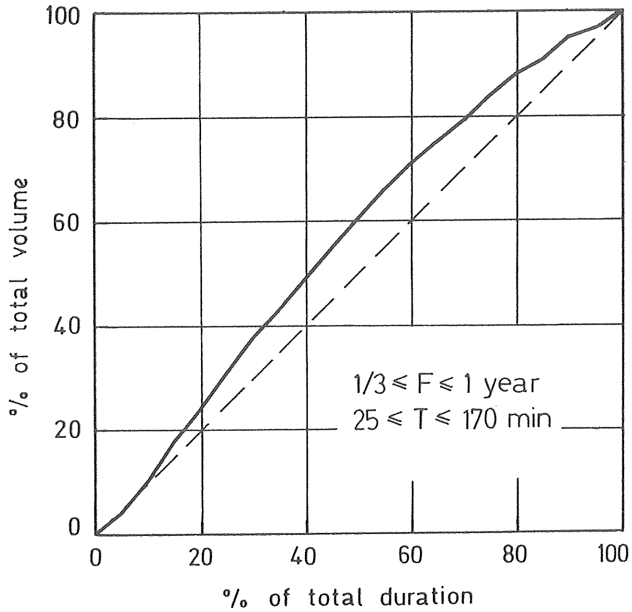


Fig. 3.9a *Average curves showing the cumulative precipitation as a function of the cumulative storm time within the maximum average intensity period. The dashed line, slope 1:1, is shown for comparison. Data from Lundby, Göteborg, 1921–1939.*



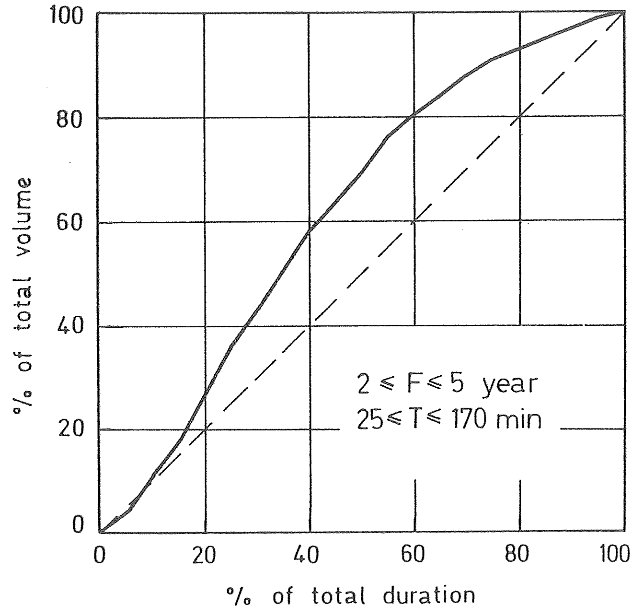


Fig. 3.9b *Average curves showing the cumulative precipitation as a function of the cumulative storm time within the maximum average intensity period. The dashed line, slope 1:1, is shown for comparison. Data from Lundby, Göteborg, 1921–1939.*

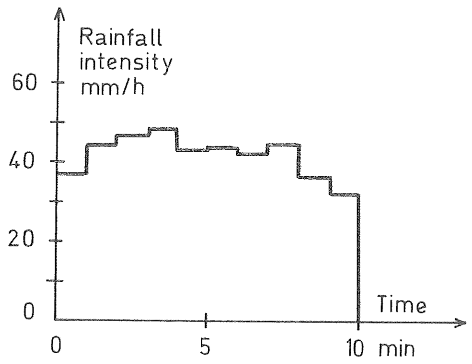


Fig. 3.10 *Example of a design storm of the same type as the Illinois State Water Survey Storm estimated from data for Lundby, Göteborg, 1921-1939. Return period one year.*

3.6 Flood Studies Report (FSR) Design Storm

General Description

In the United Kingdom design storms have been used in connection with urban runoff models since the TRRL-method was introduced into engineering work by Watkins (1962). Now, a design storm (see Fig. 3.11) described in Flood Studies Report (Natural Environment Research Concil, 1975) has been recommended for practical use. Comments on the development and use of the storm profiles have been given by Keers (1977), Keers and Wescott (1977), Folland (1978), and Kidd and Packman (1980).

The total volume of precipitation of the design storm is obtained from the Intensity-Duration-Frequency curves. Recommended total duration is 2-3 times the time of concentration. Time of concentration is the time needed for the water to move from the most remote part of the area to the point of interest. The temporal rainfall distribution is then obtained from graphs, exemplified in Fig. 3.12, or tables. The storm has its peak at the center, and the rain volume is distributed around the peak according to the curves. Different curves are given for different probabilities of obtaining a more or less peaked storm (see Fig. 3.12). For most applications the 50%

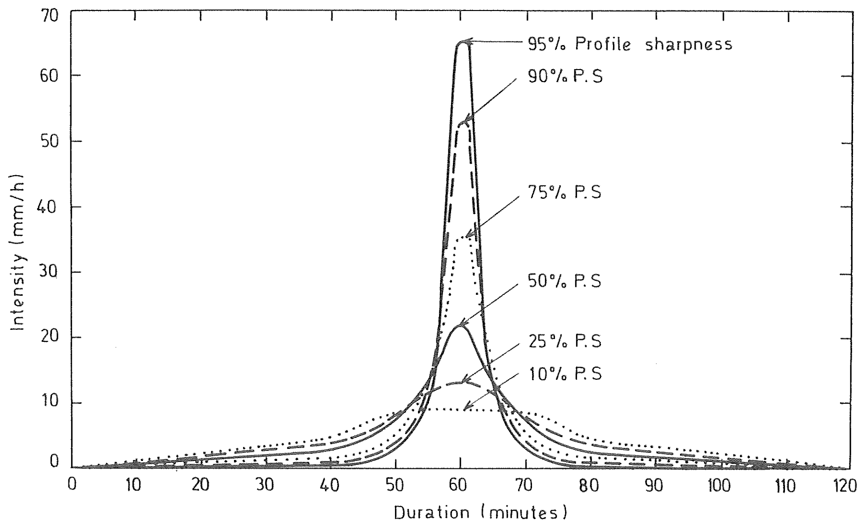


Fig. 3.11 *FSR-storm profiles for different probabilities of peakedness. Duration 2 hours and return period one year (after Keers, 1977).*

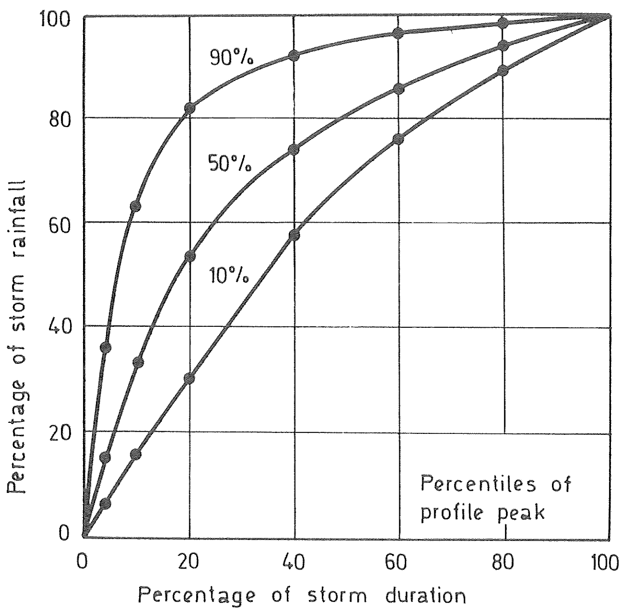


Fig. 3.12 *Cumulative percentage rainfall in England (May to October) as a function of rainfall duration. The duration, expressed as a percentage of the total duration is centered around the peak intensity. The 90%-curve means that 90% of the rainfalls are less peaked than that curve (Natural Environment Research Council, 1975).*

storm is recommended, which has been shown by Kidd and Packman (1980) to give estimations of peak-flow values comparable to peak-flow values estimated for historical storms.

For the evaluation of the design storm, 80 summer storms (May to October) and 32 winter storms (November to April) with a duration of 24 hours were analyzed. The 80 summer storms were ranked into four quartiles with 20 storms in each according to the proportion of the central 5-hour rainfall to 24-hour rainfall. The different quartiles display the difference in peakedness from sharp to flat profiles. For each quartile the 20 storms were centered on the shortest duration which gave at least 50% of the rainfall, and the mean rainfall for each hour was obtained as a percentage of the centered 24-hour volume. The result is shown in Table 3.6. Other percentiles of peakedness were then interpolated from the values in the table.

The variation in profile with storm duration was also studied. No significant differences were found when the total duration varied between 60 min and 4 days. Nor were any significant variations found in the profiles for different return periods.

Table 3.6 *Cumulative percentage rainfall (summer 24 hour storms) for the four quartiles of profile peakedness for varying ranges of duration about the profile peak. After the Natural Environment Research Council (1975).*

Cumulative duration about center		Quartile of profile peakedness			
		1	2	3	4
Hours	Percentage	Cumulative percentage rainfall			
1	4.2	6.6	10.3	22.1	35.0
3	12.5	20.2	32.2	45.7	68.6
5	20.8	33.5	48.5	63.8	80.9
7	29.2	50.1	59.9	72.6	85.8
9	37.5	61.9	65.9	78.9	89.7
15	62.5	77.4	83.0	90.9	96.1
21	87.5	91.5	95.7	96.4	99.4
24	100.0	100.0	100.0	100.0	100.0

Characteristics of the Storm

The total volume of the FSR-design storm is obtained from I-D-F curves which means that the storm does not include the total volume of the real rainfall. This is compensated for by the recommendation to use a rather long total duration. For the design of retention storages and the calculation of runoff volumes, however, the use of the FSR design storm may lead to an underestimation. Further studies are needed, which is also pointed out by Folland and Colgate (1978).

In a recently published report, Kidd and Packman (1980) (or Packman and Kidd, 1980) have compared the calculation of peak flows for the FSR design storm with calculated peak flows for historical storms. The statistical distribution functions for the two sets of peak flows were compared, and parameters of the rainfall profiles as well as an antecedent wetness index governing the percentage of runoff were varied. The final conclusions for the practical application of the FSR design storm were: a) The rainfall return period shall be equal to the required flow return period, b) The median (50%) profile for summer conditions should be used, c) The design of the pipes should be done for the total rainfall duration that gives the largest peak flows, and d) The antecedent wetness index should be obtained from a special graph. The study by Kidd and Packman (1980) shows the necessity of developing the design storms in connection with runoff simulations, sensitivity analysis of rainfall parameters, and statistical analysis.

For many applications, for example preliminary design and rough calculations, the Transport and Road Research Laboratory (1976) recommend a standard profile given in the report "Road Note 35, second edition". Folland (1978) stated that

> "The use of the profiles (in Flood Studies Report) in compiling the Table (in Road Note 35) ensures that the mean rainfall over any duration less than two

hours will have the appropriate return period listed against it. The Tables are also constructed so that the mean peakedness of the storms of durations between two minutes and two hours correspond to about a 50% storm. The actual peakedness varies greatly, however, from a zero % storm for the two minute storm to an over 90% storm for the two hour storm".

This, however, implies that the storm given in Road Note 35 is of the same type as the "Chicago design storm", with the same characteristics (see Section 3.3).

Local FSR Design Storm

Data from Lundby, Göteborg, 1921-1939, were used to evaluate a design storm of the same type as the one described by the Natural Environment Research Council (1975).

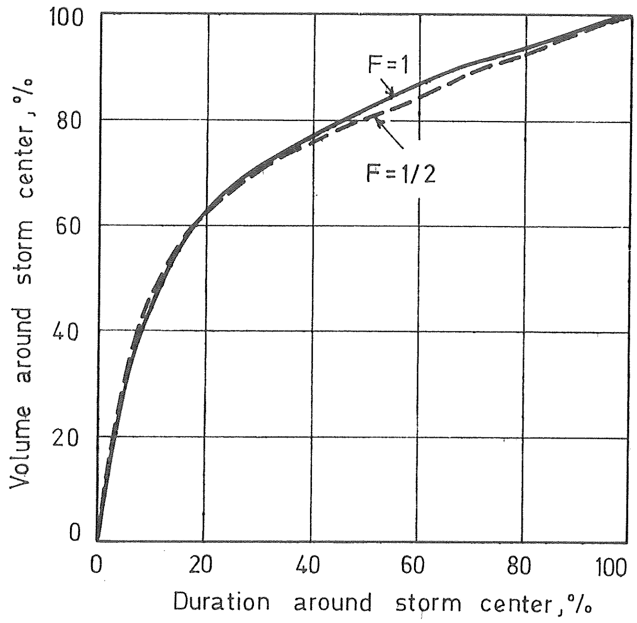
The evaluation was carried out for the part of the historical rainfalls included in the maximum average intensity part with a duration of 240 minutes. Each rainfall was centered around the middle of the duration, which gave a rain volume of approximately 50% of the volume for the 240-minute duration. The volume accumulated symmetrically around the center was then evaluated as a function of increasing duration around the center. The accumulated volume and accumulated duration were expressed in percent of the total 240-minute volume and in percent of the total 240-minute duration. The duration was increased in steps of 5 minutes on both sides of the center, and when one of the limits of the 240-minute duration was reached, the duration was increased in steps of 10 minutes towards the other limit. For historical rainfalls with a total duration less than 240 minutes, the total duration was used instead of 240 minutes.

The evaluation was carried out for the historical rainfalls with at least one maximum average intensity value, for durations from 5 minutes to 240 minutes, above the values corresponding to the values for the return periods 1/2, 1, 2, and 5 years. An average curve was evaluated for each return period and the results are shown in Table 3.7 and in Fig. 3.13. Average curves were calculated for the return periods of 1/2 and 1 year, and 2 and 5 years, respectively, because they were found almost coincide (see Table 3.7 and Fig. 3.14).

The resulting average curves were then used to construct rainfall hyetographs for different return periods and

Table 3.7 Cumulative percentage of rainfall as a function of cumulative duration about storm center for different return periods. Data for Lundby, Göteborg, 1921–1939.

Cumulative duration about center		Cumulative percentage of volume for different return periods in year					
min	percentage	1/2	1	2	5	Average of 1/2 and 1	Average of 2 and 5
10	4.2	26.7	24.5	29.6	35.1	25.6	32.3
20	8.3	42.8	40.1	48.2	54.2	41.4	51.2
30	12.5	52.1	50.4	59.2	63.6	51.2	61.4
40	16.7	58.1	58.0	66.7	69.7	58.1	68.2
50	20.8	63.0	63.6	71.2	73.0	63.3	72.1
60	25.0	66.4	67.5	73.7	75.0	66.9	74.4
70	29.2	69.5	70.8	76.9	77.5	70.2	77.2
80	33.3	72.3	73.6	79.0	79.0	72.9	79.0
90	37.5	74.7	75.8	81.0	80.6	75.2	80.8
100	41.7	76.7	77.8	82.3	81.7	77.2	82.0
110	45.8	78.6	80.1	83.8	83.6	79.4	83.7
120	50.0	80.2	82.0	84.9	84.6	81.1	84.8
130	54.2	81.8	84.0	86.5	86.4	82.9	86.5
140	58.3	83.7	86.2	88.4	88.8	85.0	88.6
150	62.5	85.6	87.8	89.7	90.0	86.7	89.8
160	66.7	87.6	89.7	91.2	90.7	88.6	91.0
170	70.8	89.1	90.8	92.1	91.7	89.9	91.9
180	75.0	90.8	92.0	93.2	93.0	91.4	93.1
190	79.2	92.2	93.3	94.0	93.9	92.7	94.0
200	83.3	94.0	94.9	95.3	95.1	94.5	95.2
210	87.5	95.7	96.5	96.6	96.5	96.1	96.6
220	91.7	97.2	97.8	98.0	98.1	97.5	98.0
230	95.8	99.0	99.4	99.3	99.3	99.2	99.3
240	100.0	100.0	100.0	100.0	100.0	100.0	100.0



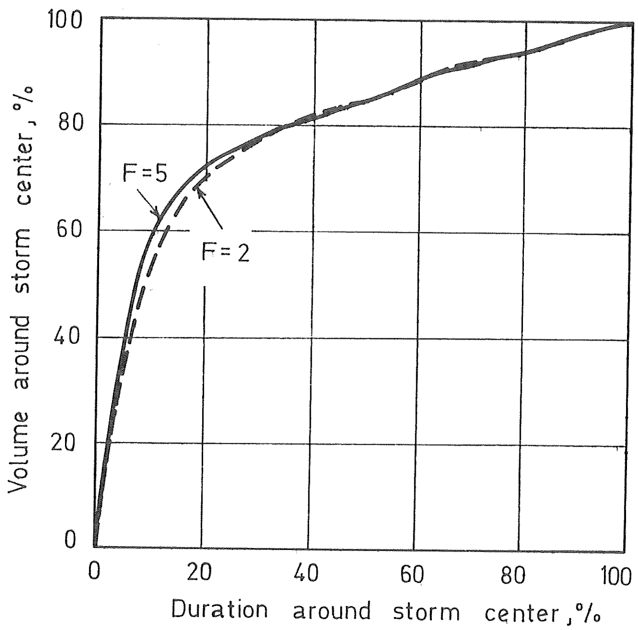


Fig. 3.13 *Cumulative percentage of rainfall as a function of cumulative duration about the storm center. Average curves for the return periods of 1/2, 1, 2, and 5 years. Data for Lundby, Göteborg, 1921-1939.*

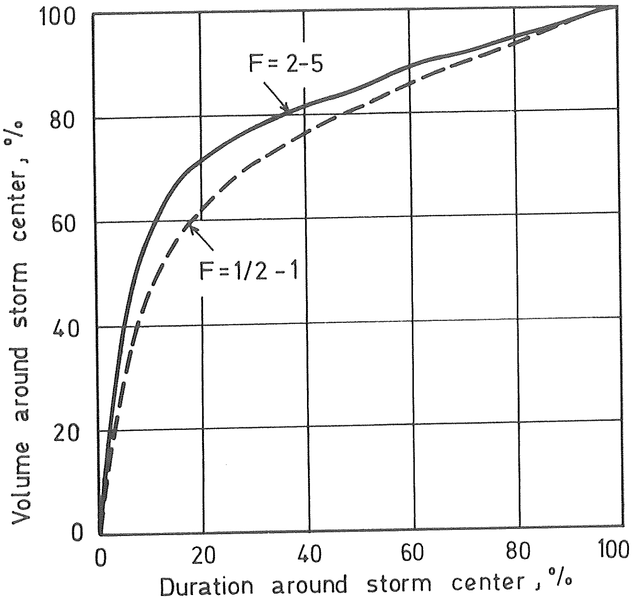


Fig. 3.14 *Cumulative percentage of rainfall as a function of cumulative duration about the storm center. Average curves for the return periods of 1/2 and 1 year, and 2 and 5 years, respectively. Data for Lundby, Göteborg, 1921-1939.*

for different total durations of the rainfalls. The total volumes of the rainfalls were obtained from the I-D-F curves. An example of a storm is shown in Fig. 3.15.

A comparison of the profile for the original FSR design storm with the profile for the local design storm for Lundby (see Fig. 3.16) shows that the local storm is more peaked. This causes large overestimations of the peak-flow values, as is shown in Section 7.3. One possible explanation for this is that the total duration is of importance for the resulting local profiles, even if no influence was reported by the Natural Environment Research Council (1975). The total duration can be taken into consideration by making the evaluation of the profiles also for different durations in the same way as was made for the local Illinois State Water Survey design storm (see Section 3.5).

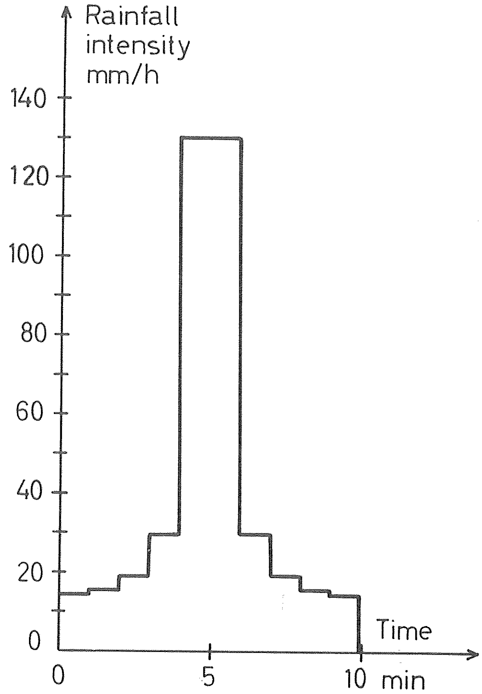


Fig. 3.15 *Example of a design storm of the type described by the Natural Environment Research Council (1975) and evaluated from data for Lundby, Göteborg, 1921-1939. Return period one year.*

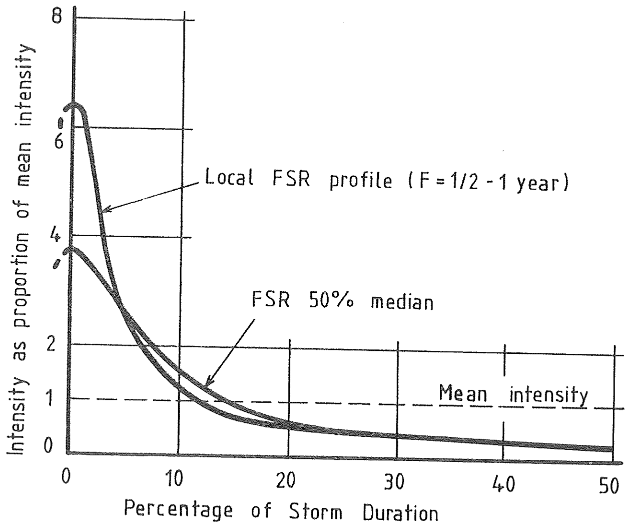


Fig. 3.16 *Comparison of the original FSR design storm, 50% profile, with the local FSR profile for Lundby, Göteborg.*

3.7 French Design Storm

Desbordes (1978) reported the development of a design storm used in connection with a storage runoff model. The main structure of the storm is shown in Fig. 3.17 and is the result of a statistical evaluation of 50 years of rainfall data at the Mountpellier - Bel Air Station.

The design storm is defined by three parameters (see Fig. 3.17):

- o The maximum average rainfall intensity i_m (T) during the intense period T of rainfall, critical for the urban catchment under consideration.
- o The maximum average rainfall intensity i_m (4h-T) during the period 4 h - T.
- o The time position of the intense period T over the whole duration of 4 hours.

The three variables were found to be statistically independent, and the parameters i_m (T) and i_m (4h-T), respectively, were exponentially or log-normally distributed. For a specified location, the values can be obtained from Intensity-Duration-Frequency relationships.

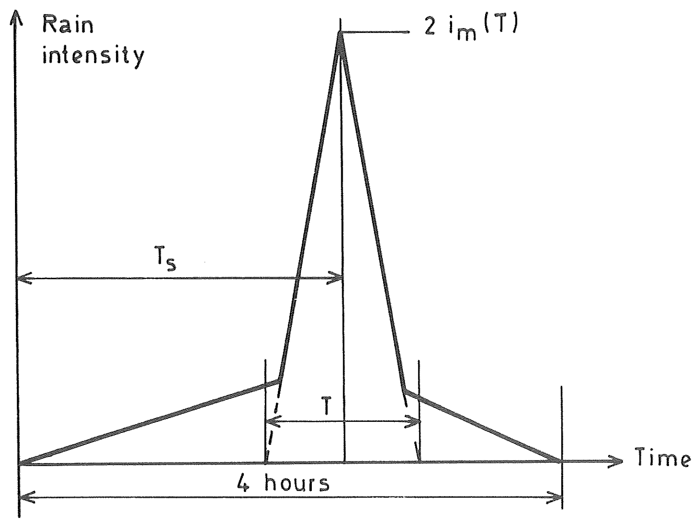


Fig. 3.17 Design storm after Desbordes (1978).

The time position T_S of the intense period T was uniformly distributed, and the value should be determined by sensitivity analysis by means of a runoff model. It was found to be an important parameter. The storm was given a total duration of 4 hours as a result of studies where the values of the peak flows were found to be unaffected if the duration was beyond 3 or 4 hours. The model runs showed that there exists a critical storm duration T which gives the maximum peak discharge. Triangular and exponential shapes of the hyetograph gave approximately the same runoff response.

The return period of the French design storm is coupled to the return periods of the I-D-F relationships through the parameters i_m (T) and i_m (4h-T). These return periods may be corrected after comparison of the simulated runoff for these design storms with the simulated runoffs for a series of historical storms. The design storm described by Desbordes (1978) is one of the few design storms that have been developed in connection with runoff simulations and sensitivity analysis of the influence on simulated runoffs of changes of the parameters describing the storm.

No local French design storm has been tested.

3.8 Concluding Remarks on Design Storms

The design storms described in Sections 3.2-3.7 were all developed at other places and reported in literature. No attempt has been made to develop a Swedish type of design storm, because the hypothesis was that the use of historical storms was superior to the use of design storms. Therefore, the aim was only to prove this by comparison of simulations of peak-flow values for existing design storms with simulations of peak-flow values for historical storms.

If the aim had been to develop a Swedish type of design storm, it could have been done in the following steps.

1. Identification of the historical storms that cause

peak-flow values close to the return periods under consideration.

2. Analysis of the rainfall characteristics of these historical storms important for the characteristics of the runoff hydrographs and identification of the parts of the rainfalls important for the runoff simulations.

3. Outline of the general form of the design storm.

4. Estimation of values of parameters of the design storm by statistical analysis of the, under item 1, identified historical storms, and by comparison of runoff simulations for the design storm with runoff simulations for historical storms including sensitivity analysis of the design storm parameters and adjustment of the parameters so the results of the two runoff simulations fit each other.

The different local design storms evaluated are compared in Fig. 3.18. All the storms are in one way or another connected to the Intensity-Duration-Frequency relationships. The I-D-F design storm is directly obtained from the I-D-F relationships. The total volume and the shape of the Chicago design storm are connected to the I-D-F curves. The total volumes of the ISWS design storm and the FSR design storm are obtained from the I-D-F relationships, which also give the volume of part 2 of the Sifalda design storm. The main difference between the design storms is the distribution in time of the rainfall amounts obtained from the I-D-F relationships. The most peaked one is the FSR design storm and the second most peaked one is the Chicago design storm. As will be shown in Chapter 7 and Appendix III, this affects the simulated peak-flow values. The more peaked the design storm is, the larger peak-flow values are obtained in the runoff simulations.

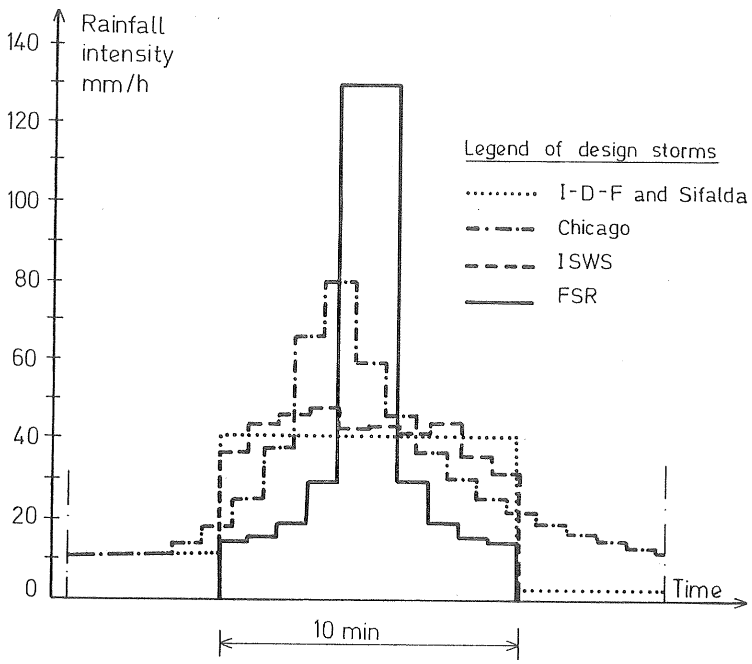


Fig. 3.18 *Comparison of the evaluated local design storms. Return period one year.*

4. HISTORICAL STORMS

4.1 Design by Statistical Analysis of Simulated Flows

When historical storms are used for the design of storm sewer systems, a number of real storms are run through a mathematical runoff model, which generates the corresponding flow-parameter values such as peak flows and runoff volumes. The statistical analysis is then applied to the flow parameters, and the values coupled with the design return interval are determined.

A complete statistical analysis, where the greater part of the statistical distribution function for the flow parameter is determined, requires that a large number of storms are run through the model to ensure that no flow values are missed. This makes the method expensive and time consuming.

Usually flow values for only one or two return intervals are of interest for the design. That means that it is necessary to determine only a part of the complete statistical distribution function. A limited part of the flow-parameter distribution function can be estimated if the historical storms corresponding to the flow values of the part of the distribution function can be identified and run through a runoff model. This makes it impossible to fit a mathematical distribution function to the points, because of the limited number of flow values, but when the curves are not extrapolated beyond the simulated return intervals, the plotted points themselves are the best statistical information of the flow-parameter values. In this case, the runoff for only a limited number of storms need to be simulated, making this method less expensive than using all the storms. However, to screen the continuous historical storm record to find the storms that correspond to the required return intervals of the flow parameters is a difficult task.

Research on the use of historical storms for the design of sewer systems is focused on the screening of continuous rainfall series and the choice of a reduced number of rainfall events. In Section 4.2, a literature review dealing with the use of historical storms is given.

4.2 Selection of Historical Rainfall Events for Runoff Simulations

For the selection of a limited number of historical storms, the following criteria should be included.

- o The chosen group of rainfalls must be large enough to ensure that the result of the statistical analysis of simulated flow-parameter values will be correct, i.e., that the right flow value is coupled with the design return period.

- o Selection of the group of rainfalls should be done in a simple way, preferably directly from the continuous rainfall record.

One method of selecting the important historical rainfall events is to carry out a statistical analysis of rainfall parameters governing the simulated peak flows. Such parameters are total rainfall depth and maximum average intensities for different durations. Other parameters can be the duration of the rainfall, antecedent moisture conditions, and different shape parameters.

Marsalek (1977, 1978a, 1978b) selected the rainfalls from a 15-year record in two steps. First, all events with a total depth larger than 12.5 mm or a maximum average intensity during 10 minutes larger than 15 mm/h were picked out. This gave 54 storms. Then the storms with the 20 largest average intensities during 5, 10, 15, 30, and 60 minutes were identified. After this process only 27 storms were left for the runoff simulations.

After the simulation of the peak flows in one real and three hypothetical catchments, Marsalek investigated the efficiency of the process of selecting historical storms. He calculated the value of the Spearman rank-correlation coefficient (Siegel, 1956) of the correlation between the ranks of peak rainfall intensities for individual durations and the associated runoff peak flows. The values of the coefficients were found to be larger than 0.545, which indicates that the correlation is significant at a level of confidence of 1%. Marsalek also calculated the value of the linear correlation coefficient between the simulated peak flows and the maximum average intensities for different durations and found the value to vary between 0.629 and 0.734. By these two correlation tests he showed that the method used for the selection of historical storms was an appropriate method.

Arnell (1978) compared the use of historical storms with the use of design storms in calculating peak flows for different return periods. He selected the historical rainfall events from a ranking list of maximum average intensities for a duration corresponding to the time of concentration of the runoff area. The time of concentration was estimated by means of the runoff model for a constant rainfall intensity. After simulation of the runoff for the selected rainfalls, the statistical distribution function of the peak flows was compared with the distribution function obtained after simulation of the runoff for "all" historical storms. The two distribution functions coincide, except for shorter return periods. In the area investigated, which is a 0.154 km^2 size residential area, the method of selecting major rainfall events seemed to be suitable.

Walesh, Lau, and Liebman (1979) used a similar method as Marsalek when they selected major rainfall events from a continuous series of data for the period of 1940-1976. A major rainfall event was defined as one in which one or more hours of continuous rainfall occurred, where the volume of rainfall was equal to or greater than that associated with a two-year recurrence interval volume for any one-hour or longer portion of the hyetograph. Hourly data were used because they were available in tabular form. A total of 42 rainfall events were obtained for the

37 years of data. The selected rainfalls where then used for the development of peak discharge - probability relationships in various areas. In one area, the result of the calculations for the major events was compared with the result of a continuous simulation. The peak discharges of the single-event simulation were found to be within 10% of the peak discharges of the continuous simulation for return periods of from 10 to 100 years.

Urbonas (1979) simulated peak flows and estimated the probability distribution functions for return periods of 2 years and longer for four areas in the Denver, Colorado, region. For this work he selected 73 rainfall events having the largest recorded one-hour rainfall accumulation to represent a partial duration series for a 73-year period of data for Denver. No check was made to find out if any storms important for the statistical analysis had been missed.

In all of the above studies, the important major storms were selected with the aid of values of the maximum average rainfall intensities for one or several durations. Marsalek (1977, 1978a, 1978b) combined this with a volume criterion. All rainfall events with intensity values above a specified threshold level were selected for runoff simulations. This gave on the average between one and two events for each year of precipitation data available, or a total number of 27 to 73 events. This is a large number to use in the design of a storm-sewer system if the interest is focused on one return period only. In some cases, return periods as short as one third of a year are of interest and this would give approximately 150 rainfall events for a 30-year period of record if one includes a safety-margin.

For the selection of major events, a method is needed that limits the selected events to a small number of events necessary for the estimation of flow parameters for the desired return period. In some cases, of course, it is of interest to estimate the statistical properties for longer return periods and on these occasions the methods stated above may be appropriate. It is also possible to develop these methods to select the rainfall events with intensities above a specified level but at the same time below another specified level.

Johansen (1979) and Johansen and Harremoës (1979) suggest that a simple and thus inexpensive runoff simulation method be used to select a group of historic storms corresponding to the design return period. These storms are then to be used as input data in a more correct simulation method. The assumption is made that the order in the ranking list of the rainfalls corresponding to the ranking list of the peak flows is nearly the same when the runoff is simulated by the simple method as when it is simulated by a more correct method. The magnitude of the peak-flow values may change, but not the order in the ranking list. Johansen has tested a linear time-area method, which is fast and inexpensive to use.

Johansen and Harremoës suggest the following procedure for the selection of the historical storms of interest.

1. The return period T is chosen.
2. Design points in the sewer system are selected.
3. For each selected design point,
   a) a time-area curve is estimated
   b) a reduced group of storms is identified from the continuous rainfall record with the aid of earlier investigations as well as the return period and the time of concentration.
   c) the simple time-area method is applied to the reduced group of storms
   d) peak flows and the corresponding rainfall numbers are ranked according to decreasing peak flows
   e) a group of representative storms are found, defined as those producing a peak flow differing from the peak flow corresponding to the return period by at most ±p% (suggestion: 5-10%).

4. By comparing the groups of storms for each design point, one or more "design storms", applicable to different parts of the area, are selected.

Steps 3e and 4 mean that a chosen deviation from the "true" value is accepted and that this deviation is small in comparison with other uncertainties involved in the design process. An example is given in Table 4.1, where the underlined storms were used in the final design of the pipes by a more correct runoff model. As can be seen from the table, only two storms (No. 884 and 987) were used in the final design.

In the method presented by Johansen (1979) and Johansen and Harremoës (1979) the assumption is made that it is so expensive to run a good computer runoff model that the number of storms used in the design must be reduced to a minimum. Compared with other costs in the design process, however, the computer costs are small, and it is not

Table 4.1 Rainfall numbers of the representative groups of storms with peak flows differing from the peak flows of the 2-year return period by at most ± 5%. After Johansen (1979). The underlined storms were selected for the final design.

Design point in the sewer system/Time of concentration (min)					
A/10	B/20	C/30	D/40	E/50	F/60
576	928	<u>884</u>	749	44	44
591	593	928	130	906	<u>987</u>
774	<u>884</u>		987	593	593
<u>884</u>	319		<u>884</u>	886	1103
476	774		276	<u>987</u>	749
903	464		886	749	660
130	209			464	947
947	627			928	515
	625			947	130
	591			130	5

necessary to reduce the number of storms as much. Moreover, the storms included in the permitted deviation of ± p% of the computed peak flows are selected by the simple time-area method and one cannot be sure that this gives the same result as a more correct method, especially if the parameters in the time-area method are estimated without accurate calculations.

A possible development of the method could be to start the design process by estimating input data for the advanced runoff model since this must be done sooner or later. A "correct" time-area curve is then calculated with the advanced model for a representative constant rainfall intensity. This time-area curve or unit hydrograph is then used when selecting the final group of design storms. Furthermore, the strict selection of one storm within ± p% from the "true" value is not made, but 1-3 storms are selected at each design point.

Design using the method of unit hydrographs for the selection of historical design storms has been compared with design using design storms (see Section 6.5 and Chapter 7).

5. DESCRIPTION OF THE RAINFALL MEASUREMENT STATION AND THE RAINFALL DATA

5.1 Description of the Rainfall Measurement Station at Lundby, Göteborg

The rainfall studies described in this report are based on data from Lundby, Göteborg (see Fig. 5.1). The place is located close to the Göta river and about 2 m above sea level. The measurements were performed during the period from 1920 to 1955.

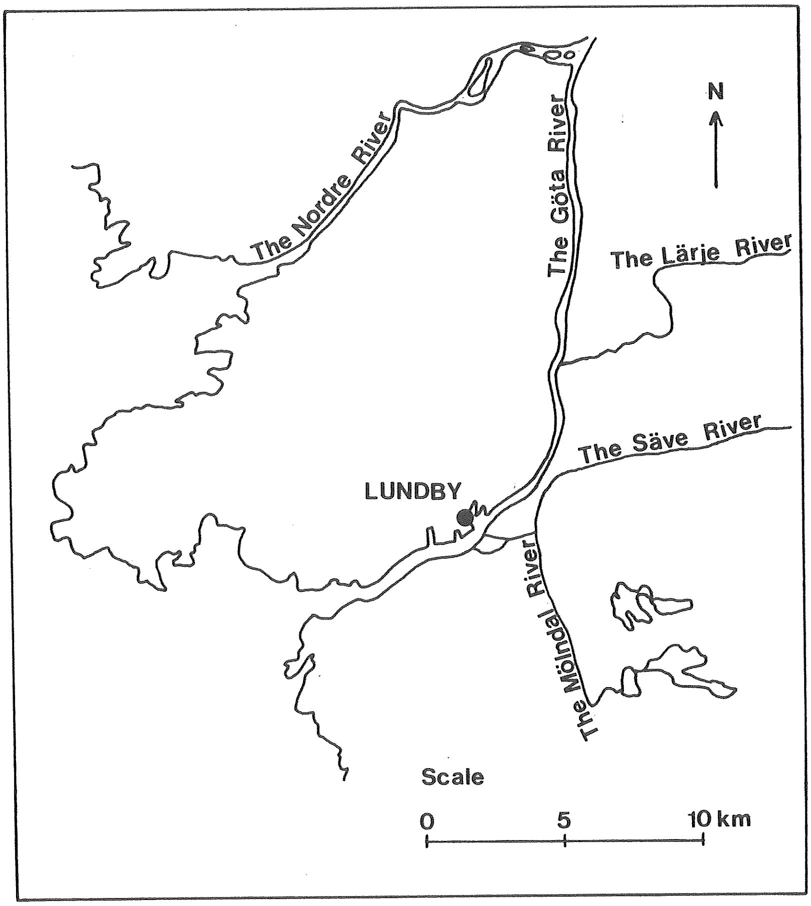


Fig. 5.1 Location of the rainfall instrument site at Lundby, Göteborg.

The instrument used was of the siphon type. Raindrops were collected in a funnel and were led to a floating chamber. When it rained, the floater rose vertically, and a curve was drawn on a moving paper by an ink-pen connected to the floater. The curve shows the accumulated precipitation as a function of time. The height of precipitation was magnified about 8 times on the graph paper, because the area of the floating chamber was smaller than that of the collecting funnel. The speed of the diagram paper was about 32 mm/h. At a rainfall height of 20 mm, the floating chamber was emptied through a siphon.

During the winter the instrument was equipped with a heating lamp to prevent it from freezing, which, besides, also resulted in snow being melted and registered as rainfall. It should be pointed out that snow measurements with this type of instrument may include great errors.

The precipitation can be evaluated at about 1/10 of a millimeter every second minute.

5.2 Evaluation of the Rainfall Data

The data available were for the period from 1920 to 1955. However, for financial reasons only data for the period from 1921 to 1939 were evaluated. The years of 1920 and 1922 were excluded because of longer periods of missing data. Thus, the total number of years treated is 18.

The rainfall data were transferred to paper tapes by means of a so-called digitizer. The data were then processed by computer and errors were corrected, after which the data were stored on magnetic tape. The time increments of the material stored varies, since only the break points of the curves were digitized. For high rainfall intensities the time increments are 1-2 minutes.

The resulting data material may be marred by different errors and uncertainties such as

- measurement errors - difficulties in estimating the true precipitation

- errors due to instrument defects

- errors introduced when transformning the data from the diagrams to the computer

- missing data

- representativeness of the period treated and the site of the instrument

- amounts of snow precipitation

The different errors have been described by Arnell and Asp (1979). The resulting error for single high-intensity values can be as high as 20-30%, but the average error is much less. When the CTH-model was tested (Arnell, 1980), the same type of rainfall instrument was used and the data were treated in the same way. The tests did not show any large errors in simulated runoffs that could be explained by errors in the rainfall data. Snow precipitation has no effect on the design studies in this report since the heavy rainfalls occur during the summer and during the autumn (see Fig. 5.2).

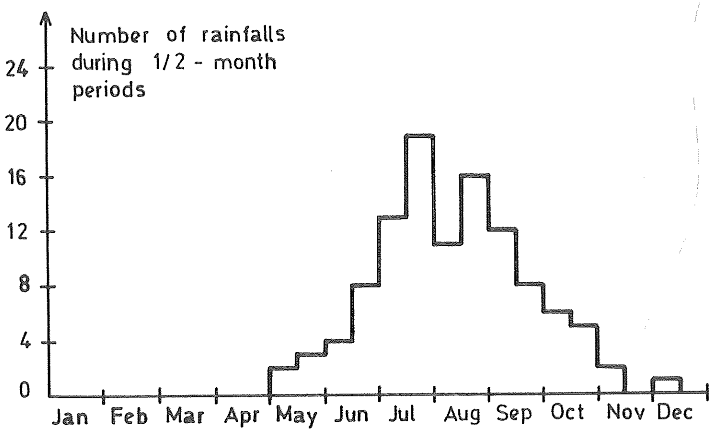


Fig. 5.2. The distribution during the year of the 54 most intense rainfalls in any of the ranked lists of maximum average rainfall intensities with durations from 5 minutes to 240 minutes.

This report contains a comparison of various sewer-system designs, for different rainfall data. All design storms and historical storms were estimated from the same time series of precipitation, so they are all marred by the same errors and biased in the same way. Potential errors are thus assumed not to influence the result of the comparison of the designs for different storms.

5.3 Selection of Independent Rainfall Events

The CTH-model used in this report is not capable of simulating the runoff continuously. The time series of rainfall must therefore be divided into individual events. Each event must also be statistically independent from the other events since one simulated characteristic flow value for each rainfall event is used in the statistical analysis of the flow values.

In Section 1.3 the time between independent rainfall intensity values was estimated to be 4 hours. That time was used as the shortest rain-free period between two independent events. A rainfall event was defined as a series of rainfall intensity values where

a) the intensity values $\geq$ 0.1 mm/h

b) intensity values < 0.1 mm/h were allowed during time intervals of maximum 4 hours within the rainfall

c) the total duration of the rainfall $\geq$ 2 minutes

d) the total volume of the rainfall $\geq$ 0.5 mm.

The intensity value of 0.1 mm/h was chosen as low as possible with regard to accuracy in measurement and evaluation of the data. The limits of the total duration and the total volume were included to exclude measurement errors and small rainfalls of no interest for the runoff simulations.

In all, about 2300 events were selected for the 18-year period. Table 5.1 gives some information concerning the individual events. The rainfall events were used for the estimation of the Intensity-Duration-Frequency curves described in Section 2.2 and the different design storms described in Chapter 3.

Table 5.1 Data concerning selected rainfall events for Lundby 1921-1939.

	Average value	Standard deviation
Number per year	129	16
Volume per event (mm)	5.3	6.1
Volume between events (mm)	0.2	0.3
Duration per event (min)	360	323
Time between events (min)	3717	5800

The selected events were also used for the runoff simulations for the historical storms. For that purpose, the rainfall events of interest were transformed to hyetographs with a time step of one minute. The present computer program for the CTH-urban runoff model used works with that time step. However, the accuracy of single one-minute intensity values is probably not good. A time step with a reasonable accuracy is estimated to be two minutes due to the speed of the strip chart recorder in the rainfall instrument, but the one-minute values used in the test of the CTH-model did not indicate any large errors that could be explained by errors in the rainfall data. Variations of short durations in the rain-intensity values are also smoothed out in the runoff process.

6. DESCRIPTION OF THE RUNOFF SIMULATIONS FOR VARIOUS TYPES OF RAINFALL DATA

6.1 Description of the CTH-Urban Runoff Model

Runoff simulations for different design storms and historical storms were carried out with the CTH-Urban Runoff Model. Arnell (1980) has made a detailed description of the function and validation of the model. Below is a summary of some of its characteristics.

The CTH-Model is a typical design/analysis single-event model. The structure of the CTH-Model, which is shown in Fig. 6.1, includes the processes of infiltration, surface depression storage, overland flow, gutter flow, and pipe flow.

When the model is applied, the total runoff area is divided into a number of subcatchments. Precipitation input data are given as over the area uniformly distributed rain-intensity values at constant time increments. Infiltration is calculated by Horton's equation, and the surface depression storage supply rate is calculated by an exponential relationship that permits the overland flow to start before the depression storages are filled. Overland flow is calculated according to a kinematic wave theory combined with a relationship between the outflow depth and the detention storage on the surface. Simulation of gutter flow is only a summation of the overland flow along the gutter. From the gutters the water is fed through inlets into the pipe system. The pipe hydraulic submodel works according to a kinematic wave theory called a non-linear reservoir cascade that allows a realistic attenuation to be simulated and describes the flow in a converging tree-type sewer system. The model can determine the pipe diameters but is not capable of treating backwater effects or pressurized flow. Retention basins can be analyzed and designed in a subroutine where the outflow, through an outlet of the nozzle-type, is a function of water depth.

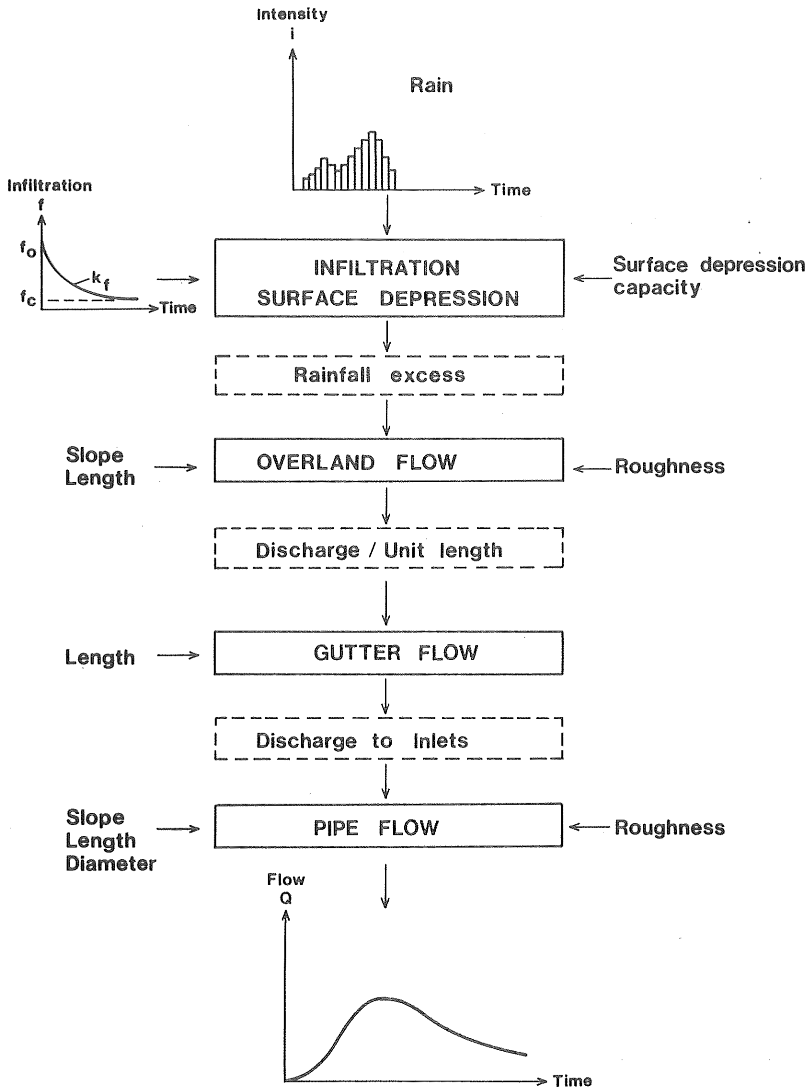


Fig. 6.1 *Structure of the CTH-Urban Runoff Model. After Arnell (1980).*

The CTH-Model was tested by simulation of the runoff for measured rainfalls in six urban areas. The test catchments are all residential areas of sizes varying between 0.035 km^2 and 1.45 km^2 and are drained by separate sewer systems. Rainfall-runoff measurements have been carried out in all areas, and for the simulations, 10-20 rainfalls were selected for each basin. Two sets of runoff simulations were carried out with different total sizes of the contributing areas and the depression storage capacities. In the first case (non-calibrated case), the sizes of the contributing areas were estimated from maps, and in the second case (calibrated case), the sizes of the contributing areas and the depression storage capacities were evaluated from rainfall-runoff data from each test basin. This was done to eliminate subjective judgment by the different persons mapping the runoff areas and writing the input data.

The agreement between the simulated and the measured runoff was, among other ways, expressed by the following numerical validation criteria: the ratios (λ) between the simulated and the observed runoff volumes and between the simulated and the observed peak flows. The values of λ were calculated for each single rainfall event, and mean values were calculated for each area together with the standard deviation of λ. A conclusive, weighted (in proportion to the number of storms for each area) average value of each validation criterion was calculated both for the "non-calibrated" case and the "calibrated" case.

For the simulated peak flows in the "non-calibrated" case, λ varied between 1.07 and 1.41 with a mean value of 1.23 and in the "calibrated" case 0.85, 1.09, and 0.95, respectively. The average value of the standard deviation of λ for the peak flows, and for the calibrated case, was 0.18. A model of the type represented by the CTH-Model is thus assumed to simulate the runoff peaks, after calibration of the runoff volumes, within an estimated model error of ± 15% and with a standard deviation of the model error of 15-20%. The term model error includes errors caused by the difficulties of estimating correct values of the model parameters and errors due to the fact that the model itself is not a perfect substitute for the urban runoff process. Further development of urban runoff models is dependent on the possibilities of improving the accuracy of the rainfall-runoff measurements, where the error now is 10-20%.

The CTH-Model was chosen for the rainfall study because it is a detailed model which has proved to make "correct" runoff simulations for measured rainfalls, and thus the model has no great influence on the conclusions concerning the choice of design storms. An example of another model that could have been used is the Storm Water Management Model (SWMM). The simulations were carried out with a time step of one minute. This was done to eliminate the reported influence for similar models of the length of the time step (see, for example, Proctor and Redfern Ltd. et al., 1976). As catchments for tests of the different types of design storms three areas, which had been used by Arnell (1980) in his validation of the model, were chosen. The same input data were used as in the validation, and thus the same accuracy can be expected.

6.2 Description of the Test Catchments

Three real catchments were used for the tests of the historical storms and the different design storms. The areas have different sizes, topography, and types of buildings as shown in Table 6.1. Arnell (1980) has described each catchment in more detail. These areas were chosen because input data were already prepared, and it is interesting to use real areas from a practical point of view.

Table 6.1 Test catchment data.

Runoff basin	Size km^2	Impermeable part %	Land use	Slopes
Bergsjön	0.154	38	Apartment complexes	Steep
Linköping 1	1.450	46	Mixed housing and commercial buildings	Flat
Linköping 2	0.185	34	Single family detached houses	Medium

The Bergsjön Basin is a 0.154 km^2 residential area with buildings consisting of three- and six-story apartment houses and a commercial building. The terrain is rather steep and rock outcrops are common. Many of the surfaces are covered with a layer of top soil often consisting of silt. Infiltration measurements show high infiltration capacities in most areas. The areas contributing to run-off are roofs, streets, pavements, and parking lots. No runoff from permeable areas was included in the simulations. The existing sewer system has a tree-like structure (Fig. 6.2) with dimensions varying between 200 mm and 800 mm. The design points chosen for the rainfall study are marked on Fig. 6.2. A summary of the input data used for simulation of the runoff for the Bergsjön basin is given in Table 6.2.

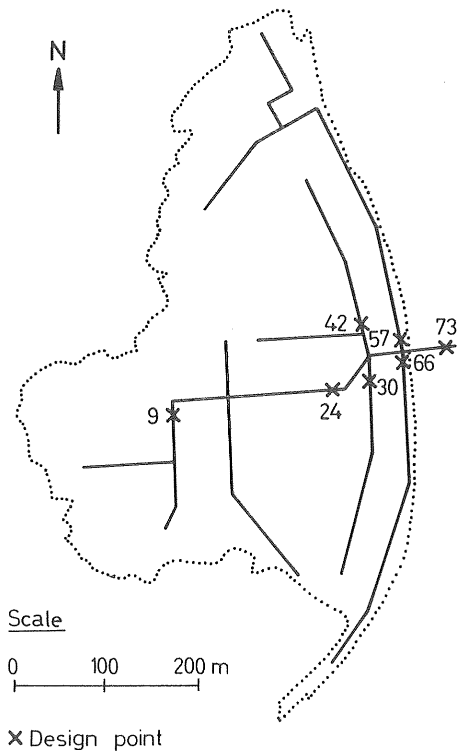


Fig. 6.2 Structure of the sewer system and location of the design points in the Bergsjön basin.

Table 6.2 Summary of runoff-simulation input data.

Input data		Bergsjön	Linköping 1	Linköping 2
Number of pipes		73	125	54
Number of inlets		47	54	49
Sizes of contributing areas	(m^2)	40 100	493 000	57 100
part of the total area	(%)	26	34	31
Surface depression storage capacity	(mm)	0.42	0.70	0.63

The Linköping 1 Basin covers 1.45 km^2, and the buildings consist of single-family detached houses and linked houses to the north, apartment complexes and commercial buildings in the center, and low industrial buildings to the south. The area is flat and most surfaces are covered with clay over glaciofluvial deposits in some parts. Although low infiltration capacities were observed in some parts of the area, no contribution from the permeable areas could be observed in the evaluation of the measured rainfall-runoff data. Therefore, no runoff from these areas was included in the runoff simulations. The areas contributing to runoff are roofs, streets, parking lots, yards, and some footpaths. The existing storm-sewer system (Fig. 6.3) consists of a main sewer with a diameter varying between 500 mm and 1 800 mm. To this main sewer the sewer systems for the different parts of the area are connected. The design points are marked on Fig. 6.3. Input data used for simulation of the runoff for the Linköping 1 basin are listed in Table 6.2.

The Linköping 2 Basin, which is located within the Linköping 1 basin, covers 0.185 km^2 and has single-family detached and linked houses and a few school buildings. In the eastern part, the area is flat and covered with clay. The western part has a more broken ground and the soil consists of

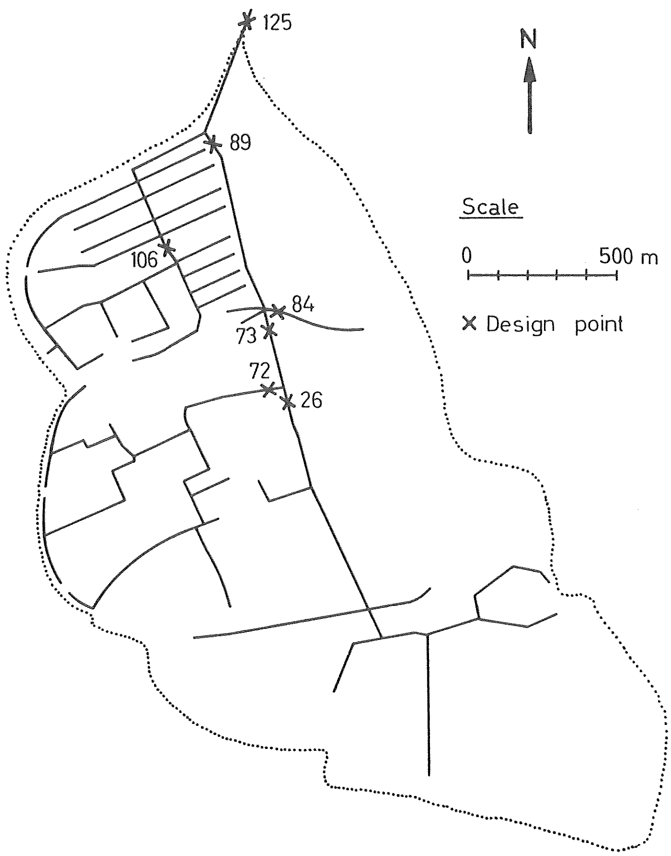


Fig. 6.3 Structure of the sewer system and location of the design points in the Linköping 1 basin.

sand and till. No inlets are draining the permeable areas, so the permeable areas were not included in the runoff simulations. Areas contributing to the runoff are roofs, streets, and footpaths. Some footpaths and driveways are drained to surrounding lawns. The structure of the storm-sewer system is shown in Fig. 6.4 together with the design points used in the rainfall studies. The maximum pipe diameter in the existing system is 800 mm. Table 6.2 gives some important data used in the runoff simulations for the Linköping 2 basin.

The input data used for the runoff simulations in the three catchments and listed in Table 6.2 were the same as

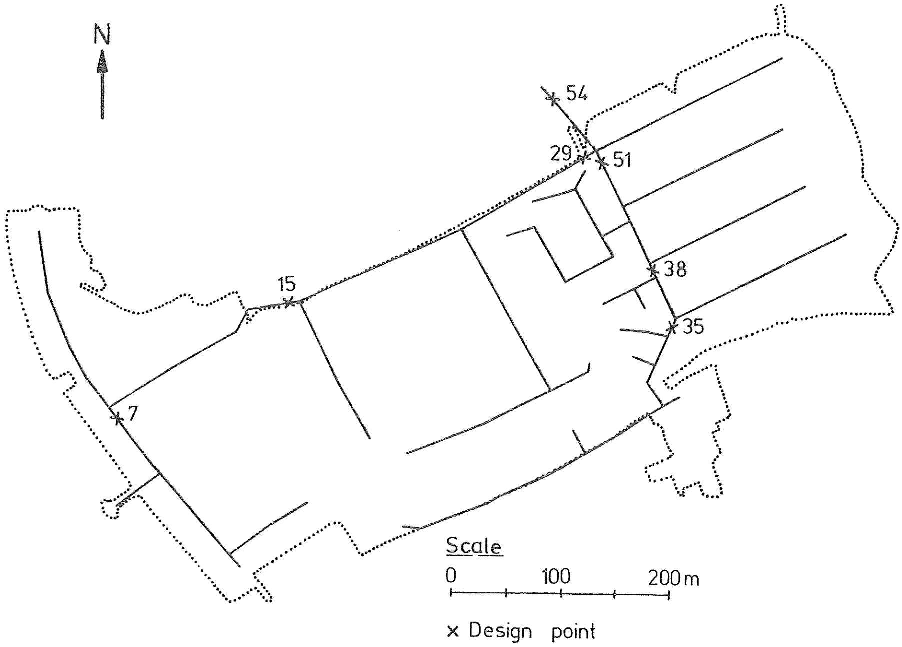


Fig. 6.4 Structure of the sewer system and location of the design points in the Linköping 2 basin.

those used by Arnell (1980) in his validation of the CTH-Model. For each catchment the total sizes of contributing areas and depression storage capacities were calibrated by a linear regression analysis of measured rainfall volumes and runoff volumes (see Arnell, 1980).

The runoff simulations were made with the assumption that no significant flow in the pipe systems originated from the permeable surfaces due to either high infiltration capacities or no inlets draining those surfaces. This is most likely a correct assumption for rainfalls with return periods of up to a few years and thus should not affect the design peak flows with normal return periods of one to two years. Lyngfelt (1981) has tested if the runoff volumes for the 40 most intense rainfalls during two years increased compared with the volumes obtained for all rainfalls. He found no increase in the sizes of contributing surfaces except for the Bergsjön basin, where the contributing part increased from 26% to 30% of the total area. For some of the most intense rainfalls used in this study, one can expect some runoff from permeable areas and thus underestimated simulated flow values for these rainfalls can be expected in the present study. This will affect the statistical distribution functions of the runoffs for longer return periods. Also, if the consequences of very high-intensity rainfalls are to be studied in a runoff area, the possibilities of runoff from permeable areas must be investigated carefully.

The influence on runoff volumes from antecedent precipitation was also ignored, since the runoff from permeable areas was neglected. This is probably not a rough approximation because no influence from permeable areas was found in the rainfall-runoff measurements (see Arnell, 1980). An attempt in the Linköping 2 basin to correlate volume runoff coefficients (baseflow separated runoff volumes divided by rainfall volumes) with antecedent precipitation during five days preceding the events gave no result. In all catchments the impermeable areas are clearly defined and most subcatchments are surrounded by curbstones. All roofs are directly connected to the sewer pipe system and no ditches are used for drainage of the basins.

The pipe flow routing was carried out with pipe diameters of the same size as in the real catchment. When the maximum peak of the inflow to a pipe exceeds the sewer capacity, the diameter is increased to a standard diameter with sufficient capacity. This will be the case when simulating the runoff for the more intense rainfalls. In a real case these rainfalls would have caused flooding of the pipes and an attenuation of the peak flows. So, the model will overestimate the peak flows for these rainfalls. In the design case when the sizes of the pipes are unknown, but the pipes are designed to carry the flow without flooding, it must be correct to simulate the flow without any attenuation caused by too small pipes.

To study the effect of using the real pipe diameters instead of using as small diameters as possible, a few test runs were made with the diameters chosen to be 225 mm in the input data. The program then increased the diameters to sizes with sufficient capacity for each storm. The Average-Intensity-Duration design storm and the Chicago design storm were used for the test runs. The simulated peak-flow values for the 7 design points of each runoff area were compared with the peak-flow values obtained for the real pipe diameters. The deviation in peak-flow values were in most cases less than 1% and as a maximum 2%. Thus, the effect of using the real pipe diameters in the simulations can be ignored.

The comparison of flow values for different types of rainfall data was done in 7 *design points* of the sewer system in each area. That means that the statistical evaluation of, for example, the peak flows was carried out for a few specified pipes in the sewer systems. However, the runoff model was validated by comparison of simulated and measured flow values at the outlet of the areas. The accuracy of the simulated flows upstream of the outlet pipe depends, among other things, on the level of discretization of the runoff areas, i.e., how many subareas are used to describe the total runoff area.

For the Bergsjön and the Linköping 2 basins, the divisions into subcatchments were detailed and water was allowed to enter the system at nearly every junction. For these catchments the simulated flow values are believed to be correct in all the pipes of the sewer system. Furthermore, no hypothetical pipes (no simplification of the pipe system) were used in the input data for these areas.

For the Linköping 1 basin rather large subcatchments were used; on the average about 9000 m^2 of contributing areas were connected to each inlet. The sewer system was simplified, and the detailed real pipe system was exchanged for one including the real main pipes and only a few hypothetical pipes in each subcatchment. This means that the flow values are probably not as accurate in the most upstream part of the pipe system as closer to the outlet. A subdivision of the catchment on this level is suitable for the design of the main pipes. Therefore, the design points of the Linköping 1 basin are located in the main pipe system. One point is equal to the outlet pipe of the Linköping 2 basin. Thus, it was possible to compare the flow values in that pipe simulated for the Linköping 1 basin with the flow values simulated for the Linköping 2 basin.

The simulated peak flows for the historical storms for pipe No. 54 of the Linköping 2 basin are shown in the statistical distribution function in Appendix III. The corresponding peak flows for pipe No. 106 of the Linköping 1 basin are shown in the same Appendix.

The peak-flow values for pipe 106 are 15-25% (counted on peak-flow values for pipe 54) smaller than the peak-flow values for pipe 54. The values for pipe 54, Linköping 2, are assumed to be more correct due to a more detailed discretization of the basin into subcatchments. The difference might explain why the peak-flow values were underestimated when validating the CTH-Model using the data for the Linköping 1 catchment (see Arnell, 1980). It also shows the difficulties in discretizing the basins. Experience shows that it is important not to oversimplify the structure of the pipe system and the number of inlets. Too little knowledge is, however, available, and the methods for discretization of the runoff areas and the pipe systems need to be further investigated.

The differences in the flow values between pipe 54 and pipe 106 are the same for the historical storms and for the different design storms. Thus, the comparison of designs for different types of rainfall data is not significantly influenced.

6.3 Runoff Simulations for Design Storms

The runoff was simulated in all three test catchments for the following types of design storms:

- o Average-Intensity-Duration storms (I-D-F)
- o Chicago design storms (Chicago)
- o Sifalda design storms (Sifalda)
- o Illinois State Water Survey storms (ISWS)
- o Flood Studies Report storms (FSR)

The simulations included the return periods of 1/2, 1, 2, and 5 years.

The different storms are described in Chapter 3, where values of the different parameters governing the storms are also given. The following additional assumptions were made when simulating the runoffs for the different storms.

o I-D-F storm — The surface depression storage was selected to be zero since rainfall prior to the maximum-average-intensity period is neglected. For each return period, runoff simulations were made for different durations to find the duration that gave the maximum peak flow.

For durations below 5 minutes, and between 5 and 10 minutes, intensity values were estimated by Eq. 2.2 and by values of the constants obtained from Table 2.3.

o Chicago storm — No extra assumptions were made.

o Sifalda storm — Runoff simulations were made for different durations of the central part ② of the rainfall to find the duration that gave the maximum peak flow.

- ISWS storm — The surface depression storage was selected to be zero. Runoff simulations were made for different total durations to find the one that gave the maximum peak flow.

- FSR storm — The surface depression storage was selected to be zero. Runoff simulations were made for different total durations to find the one that gave the maximum peak flow.

By the runoff simulations, one maximum peak-flow value was obtained for each return period and for each design point. The result is plotted on the graphs given in Appendix III. The graphs show the peak-flow values for different return periods both for the design storms and for the historical storms.

In Table 6.3 the durations that caused the maximum peak flows are listed for the different storms. The durations are short for the upstream parts of the basins. In two pipes of the Bergsjön basin a duration of 3 minutes corresponds to the maximum peak-flow values. This is a short time step compared to the time step of one minute used in the runoff calculations. It is possible that the runoff simulations in the upstream parts of the basins could have been improved if a shorter time step had been used. In the downstream parts of the sewer systems no effect of the short durations is expected. The rain intensity of measured rainfalls used in the test of the CTH-Model varied as much as the intensity of the design storms used in the present study, and no deviations between measured and calculated runoffs could be explained by the choice of the length of the time step. Furthermore, Lyngfelt (1978) has found the influence of variations of the time step, on computed peak-flow values, to be small.

Table 6.3 The total storm durations (for the Sifalda-storm duration of part ②) that caused the maximum peak flows. Return period one year. Values in minutes.

Catchment	Design point No.	I-D-F storm	Sifalda storm	ISWS storm	FSR storm
Bergsjön	9	3	3	3	15
	24	5	5	5	20
	30	3	3	3	15
	42	4	4	4	20
	57	6	5	5	20
	66	5	4	5	20
	73	5	4	5	25
Linköping 1	26	5	5	6	25
	72	9	7	9	35
	73	9	9	10	35
	84	6	5	6	25
	89	20	9	20	50
	106	9	7	9	25
	125	20	9	20	50
Linköping 2	7	4	4	4	20
	15	6	5	6	20
	29	8	6	8	25
	35	7	6	7	25
	38	6	6	7	25
	51	6	5	6	25
	54	8	6	8	25

The variations of the calculated peak flows due to the durations of some of the design storms are illustrated in Fig. 6.5. For the smaller Bergsjön basin, a significant influence on the peak-flow values was obtained, if the duration of the storm varies more than 1-2 minutes. For the larger Linköping 1 basin the influences of variations due to the durations of the storms are not so pronounced. As a conclusion, the durations should not be varied in steps longer than 1-2 minutes for small catchments.

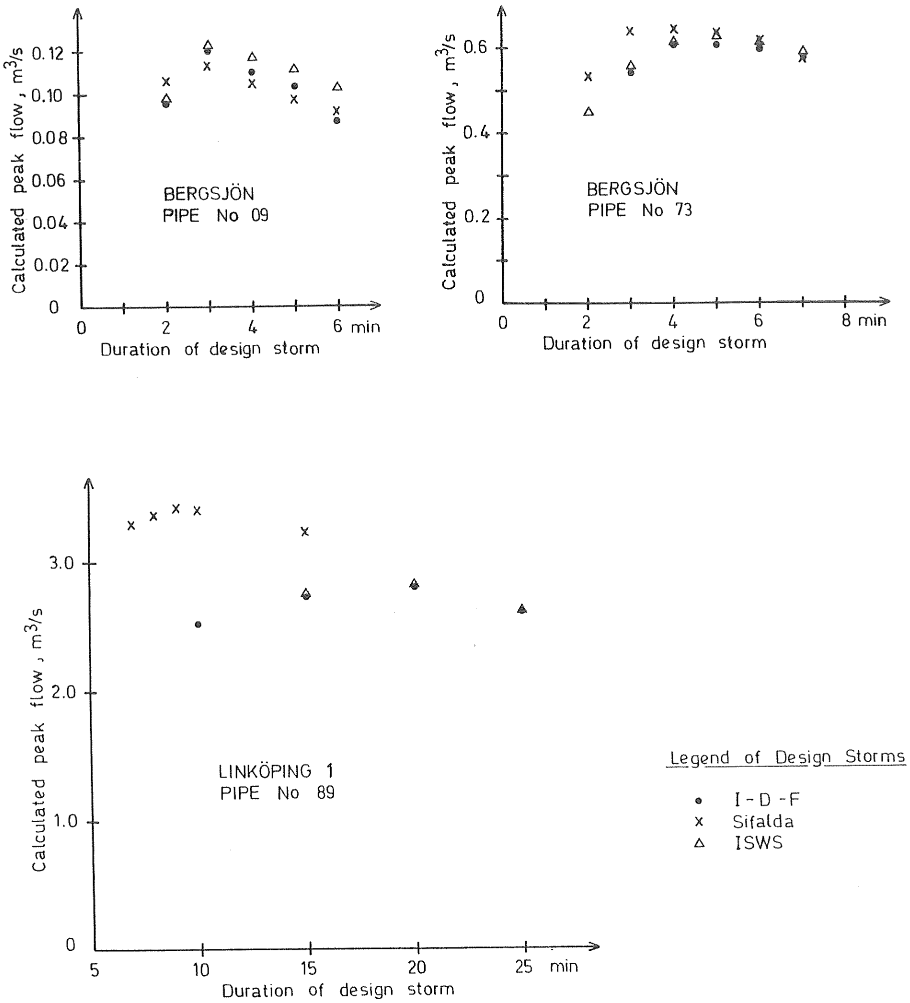


Fig. 6.5 *Sensitivity of the calculated peak-flow values to the durations of the design storms (for the Sifalda storm duration of part 2). Return period one year.*

6.4 Runoff Simulations for Historical Storms

In each test catchment the runoff was simulated for a number of historical storms. In order to obtain enough data for a statistical estimation of peak flows with a return period of 1/2 year or longer (1/2 year corresponds to approximately 36 storms since 18 years of rainfall data were used), the 54 largest storms were identified in any of the ranked lists of maximum average intensities for durations of from 5 minutes to 30 minutes for the Bergsjön and the Linköping 2 basins and to 120 minutes for the Linköping 1 basin. This gave a total number of 74 rainfalls for the Bergsjön and the Linköping 2 basins and 94 rainfalls for the Linköping 1 basin.

The peak flows were calculated for each rainfall for the chosen design points of each test basin. The 36 largest peak-flow values for each design point were ranked in decreasing order, and plotting positions were calculated for each value by Eq. (2.1). The results are shown in Appendix III.

A check was also made to see if it was enough to use the 54 largest storms for different durations to obtain a correct statistical result. It was found that almost all storms contributing to the 36 largest peak flows for each design point came from the group of the 36 largest storms in ranked lists of maximum average intensities for durations from 5 minutes to 30 minutes for the Bergsjön and the Linköping 2 basins and to 120 minutes for the Linköping 1 basin.

For the Bergsjön basin two storms for each design point (for one design point one storm only) belonged to the group of storms between the 54 largest and the 36 largest storms. For the Linköping 2 basin one storm for two design points and two storms for one design point were identified as belonging to the group between the 54 and the 36 largest storms. Similarly, one storm was found for each of two design points in the Linköping 1 basin. This means that the selected group of storms should have been larger than the 54 largest ones to give an "exact" estimation of the statistical distribution function for the simulated peak flows. However, the influences are small and appear at the lower end of the ranked lists. The effects on the resulting distribution functions are assumed to be negligible.

6.5 Runoff Simulations Using Unit Hydrographs

In order to test the method for selecting historical storms suggested by Johansen (1979), the peak flows for a number of historical storms were calculated by means of unit hydrographs. The work was done in the following steps for each design point in all three test catchments.

1. An S-hydrograph or time-area curve was estimated with the CTH-Model for a constant rainfall intensity preceded by a lower rainfall intensity so as to get a unit hydrograph that could transform changes in the rainfall intensities to changes in the runoff values.

   For the Bergsjön and the Linköping 2 basins, a constant rainfall intensity of 41.6 mm/h was used, preceded by an intensity of 4.16 mm/h. For the Linköping 1 basin the corresponding values were 28.7 mm/h and 2.87 mm/h, respectively. These rather arbitrarily chosen constant intensities correspond to durations of 10 and 20 minutes and a return period of one year for which the sewer systems were designed (see Table 2.2). The resulting S-hydrographs are shown in Figs. 6.6-6.8.

2. A unit hydrograph for a rainfall duration of one minute was estimated by first shifting the S-hydrograph one minute, and calculating the difference between the original and the shifted S-hydrographs. The unit hydrograph was then divided by the constant rainfall intensity minus the preceding intensity. A duration of one minute was used because the historical rainfall data to be used were stored with a time increment of one minute.

3. Peak-flow values were estimated for 110 historical storms by means of the unit hydrograph. These 110 storms are the 54 most intense storms in any of the ranked lists of maximum average rainfall intensities for durations from 5 minutes to 240 minutes. The peak-flow values were ranked in descending order together with the corresponding rainfall numbers.

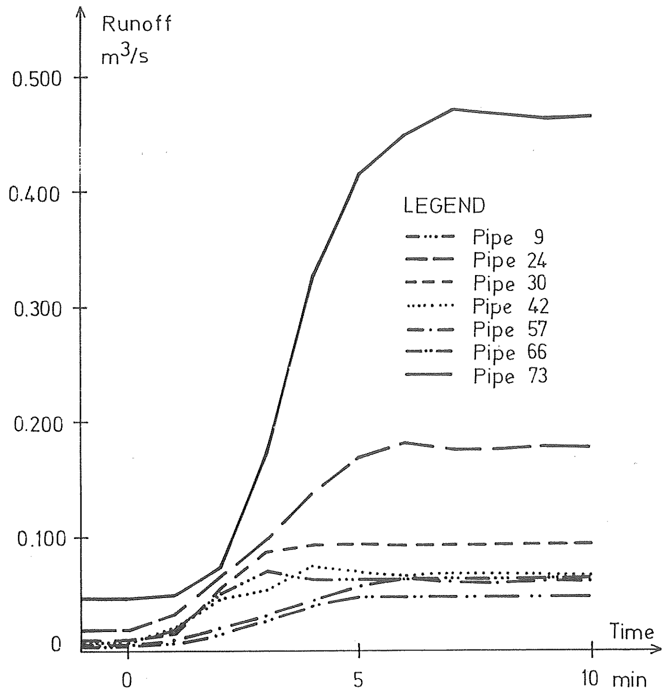


Fig. 6.6 *S-hydrographs for the Bergsjön basin for a constant rainfall intensity of 41.6 mm/h, preceded by an intensity of 4.16 mm/h.*

A model for initial rainfall losses was coupled with the unit hydrograph. No runoff was generated before the depression storages were filled. The same depression storage capacities were used as for the simulations with the CTH-Model.

4. The ranked peak flows (and rainfall numbers) for all design points in a test basin were listed in a common table together with a list of the return periods corresponding to each peak-flow value. The return periods were calculated by Eq. (2.1). The results are shown in Tables 6.4-6.6.

   For each return period of 1/2, 1, 2, and 5 years, the group of storms was identified for which the peak-flow values differed from the peak flow-value corresponding to the return period studied by at most ± 5%. The groups of storms are indicated in Tables 6.4-6.6.

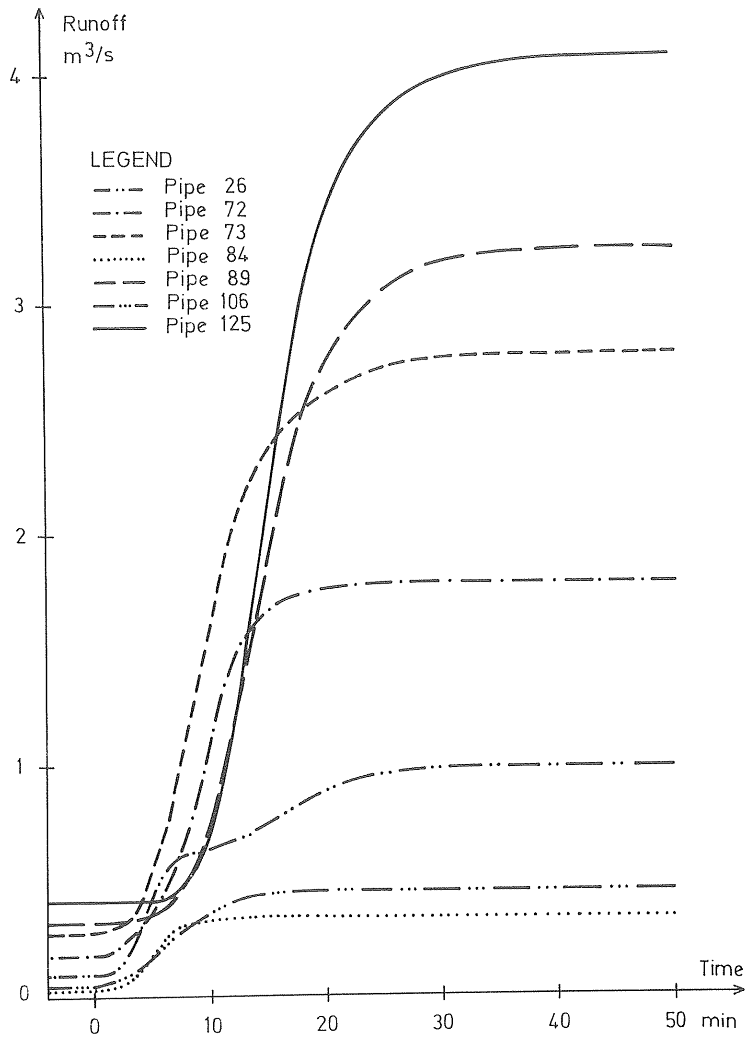


Fig. 6.7 *S-hydrographs for the Linköping 1 basin for a constant rainfall intensity of 28.7 mm/h, preceded by an intensity of 2.87 mm/h.*

The deviation of ± 5% was chosen because it was used by Johansen (1979). If the method is to be further developed, the magnitude of the deviation requires a special study.

5. By comparing the groups of storms for all design points of a basin, the storms with identical rainfall numbers for as many design points as possible were identified.

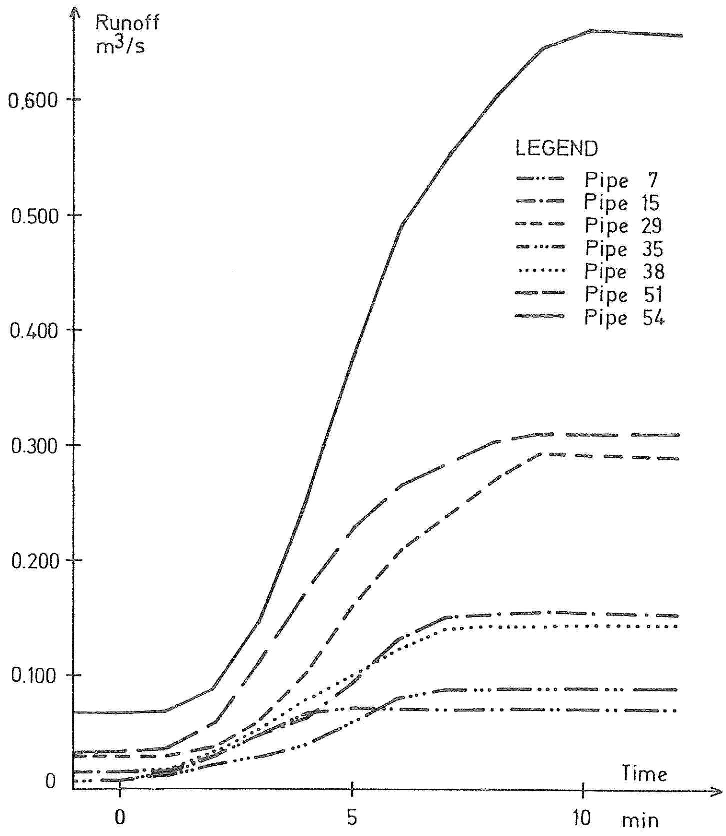


Fig. 6.8 *S-hydrographs for the Linköping 2 basin for a constant rainfall intensity of 41.6 mm/h, preceded by an intensity of 4.16 mm/h.*

Thus, a few historical "design storms" were selected for the final design. Sometimes only one storm could be used for the entire catchment. The resulting "design storms" are underlined in Tables 6.4-6.6.

6. For each return period the peak flows were calculated with the CTH-Model for the historical "design storms" (in fact the calculations were made earlier when the simulations were made for "all" historical storms). The results for all design points are marked in the figures in Appendix III.

Table 6.4 *The Bergsjön basin. Ranked peak-flow values calculated with a unit hydrograph. Marked groups represent storms with peak-flow values differing from the peak-flow value corresponding to the return period studied by at most ± 5%. Common historical "design storms" are underlined.*

No	Return period year		Rainfall numbers and Peak flow values (m^3/s) for pipe no 9		24		30		42		57		66		73	
1	32.28		596	0.3207	596	0.6153	596	0.3942	596	0.2736	596	0.2124	596	0.1745	596	1.763
2	11.88		2171	0.2276	2025	0.4511	2171	0.2892	2025	0.2205	2025	0.1620	2025	0.1291	2025	1.326
3	7.20		2025	0.2215	2171	0.4486	2025	0.2804	2171	0.2032	2171	0.1593	2171	0.1251	2171	1.279
4	5.16	5	885	0.2040	885	0.3778	885	0.2426	885	0.1799	885	0.1334	885	0.1064	885	1.136
5	4.02		1669	0.1698	475	0.3446	1669	0.2257	475	0.1613	475	0.1217	1669	0.0979	1669	0.994
6	3.29		2302	0.1689	1669	0.3391	2302	0.2127	1669	0.1563	1669	0.1213	475	0.0976	475	0.953
7	2.79		1036	0.1621	620	0.3121	1036	0.2113	759	0.1444	2280	0.1134	1036	0.0860	1909	0.892
8	2.41		1568	0.1594	2280	0.3111	475	0.2092	620	0.1383	620	0.1113	620	0.0839	2302	0.866
9	2.13	2	475	0.1585	1909	0.3089	1568	0.1994	1909	0.1381	1909	0.1072	1909	0.0839	620	0.858
10	1.91		1759	0.1564	1778	0.2970	1909	0.1935	893	0.1302	1778	0.1064	2302	0.0839	759	0.855
11	1.73		759	0.1555	1265	0.2923	759	0.1930	2280	0.1302	1265	0.1044	2280	0.0837	1036	0.848
12	1.58		1909	0.1552	759	0.2918	2280	0.1867	2302	0.1273	759	0.1029	759	0.0819	2280	0.815
13	1.45		1023	0.1540	2302	0.2869	620	0.1803	1568	0.1260	2302	0.1014	893	0.0791	1568	0.814
14	1.34		2280	0.1485	893	0.2861	1023	0.1795	1778	0.1255	893	0.1013	1568	0.0784	893	0.801
15	1.25		619	0.1397	1036	0.2829	900	0.1775	1265	0.1241	1036	0.1010	1778	0.0778	1265	0.784
16	1.17		900	0.1382	1541	0.2747	1759	0.1759	1668	0.1235	1541	0.0973	1265	0.0768	1778	0.779
17	1.10		620	0.1365	1568	0.2705	1249	0.1727	1036	0.1232	2397	0.0960	1541	0.0748	1541	0.722
18	1.04	1	1249	0.1339	1668	0.2701	893	0.1702	1541	0.1220	1668	0.0950	1668	0.0735	1024	0.712
19	0.98		893	0.1319	2397	0.2699	1024	0.1692	2397	0.1195	1568	0.0940	2397	0.0724	2397	0.709
20	0.93		1024	0.1316	1432	0.2485	1265	0.1659	1432	0.1137	1432	0.0895	1024	0.0716	1759	0.707
21	0.88		1265	0.1259	1905	0.2458	619	0.1600	493	0.1130	1905	0.0872	1432	0.0696	1249	0.705
22	0.84		1261	0.1226	1445	0.2403	1778	0.1584	1249	0.1126	1906	0.0857	1249	0.0694	1668	0.705
23	0.81		1301	0.1225	493	0.2353	1261	0.1577	710	0.1110	1445	0.0854	1905	0.0680	1432	0.695
24	0.77		1778	0.1210	1906	0.2344	2397	0.1576	1759	0.1101	493	0.0829	1906	0.0668	1023	0.680
25	0.74		1541	0.1203	1024	0.2343	1541	0.1559	1301	0.1089	1249	0.0825	1301	0.0664	1301	0.679
26	0.71		2283	0.1197	2283	0.2307	1301	0.1539	2283	0.1086	1301	0.0822	1261	0.0662	1905	0.666
27	0.68		2397	0.1196	2174	0.2306	1806	0.1523	1906	0.1076	2283	0.0810	1445	0.0651	1261	0.663
28	0.66		1806	0.1192	1249	0.2302	1905	0.1523	1024	0.1068	2174	0.0808	2174	0.0645	1906	0.663
29	0.64		1432	0.1178	1301	0.2295	2283	0.1510	1905	0.1068	1024	0.0804	900	0.0627	1445	0.652
30	0.61		1905	0.1159	1261	0.2199	1432	0.1508	1445	0.1061	1261	0.0782	2283	0.0619	619	0.649
31	0.59		1668	0.1148	1805	0.2093	1668	0.1490	2174	0.1040	1805	0.0749	1806	0.0610	710	0.646
32	0.58		710	0.1136	347	0.2086	710	0.1467	1261	0.1037	2411	0.0744	1023	0.0606	900	0.639
33	0.56		1906	0.1123	2411	0.2067	1906	0.1467	1023	0.1021	347	0.0735	710	0.0603	2283	0.636
34	0.54		711	0.1101	1806	0.2061	736	0.1416	900	0.1001	2276	0.0731	347	0.0599	493	0.625
35	0.53		736	0.1093	2276	0.2045	1445	0.1411	1806	0.0991	1806	0.0725	1759	0.0599	2174	0.623
36	0.51		1445	0.1079	710	0.2005	1912	0.1402	619	0.0982	1912	0.0716	1572	0.0581	347	0.614
37	0.50	1/2	2174	0.1078	1274	0.1991	1572	0.1389	711	0.0961	710	0.0714	736	0.0578	1806	0.614
38	0.48		493	0.1077	1539	0.1976	1268	0.1387	736	0.0956	1274	0.0708	493	0.0572	736	0.585
39	0.47		1572	0.1068	1912	0.1961	711	0.1373	347	0.0938	1917	0.0695	711	0.0572	1917	0.572
40	0.46		1912	0.1064	736	0.1960	2174	0.1365	1274	0.0931	736	0.0692	1805	0.0569	711	0.572
41	0.45		1268	0.1052	1917	0.1920	347	0.1363	1917	0.0922	1282	0.0688	1274	0.0566	1572	0.569
42	0.44		347	0.1039	1759	0.1909	1917	0.1328	2411	0.0910	1539	0.0678	619	0.0563	1805	0.565
43	0.43		1917	0.1020	1282	0.1903	493	0.1327	1572	0.0903	711	0.0664	1268	0.0563	1912	0.565
44	0.42		607	0.0995	1023	0.1895	607	0.1295	1805	0.0901	1572	0.0659	1282	0.0549	1274	0.555
45	0.41		1539	0.0995	1572	0.1894	1282	0.1270	1912	0.0895	613	0.0651	1917	0.0548	1268	0.555
46	0.40		1282	0.0991	711	0.1868	1805	0.1261	1282	0.0885	1268	0.0641	1539	0.0547	1282	0.555
47	0.39		1447	0.0969	619	0.1830	1274	0.1238	607	0.0868	900	0.0640	1912	0.0547	2411	0.547

Table 6.5 *The Linköping 1 basin. Ranked peak-flow values calculated with a unit hydrograph. Marked groups represent storms with peak-flow values differing from the peak-flow value corresponding to the return period studied by at most ± 5%. Common historical "design storms" are underlined.*

Rainfall numbers and Peak flow values (m^3/s) for pipe no

No	Return period year		26		72		73		84		89		106		125	
1	32.28		596	3.2763	596	5.9092	596	8.1593	596	1.5337	596	8.9550	596	1.4760	596	11.65
2	11.88		2025	2.4384	2025	5.1825	2025	7.1610	2025	1.1827	2025	8.0471	2025	1.2767	2025	10.42
3	7.20		2171	2.1815	2171	4.5716	2171	6.1455	2171	1.1567	2171	6.6553	2171	1.1031	2171	8.73
4	5.16	5	885	1.8748	885	4.0555	885	5.4119	885	0.9513	885	6.1856	885	0.9828	885	7.98
5	4.02		759	1.7701	1265	3.4343	1265	4.7430	1669	0.8599	1265	5.2959	1265	0.8271	1265	6.83
6	3.29		1669	1.7274	1778	3.3286	1778	4.6910	475	0.8119	1778	5.1244	1778	0.8252	1778	6.71
7	2.79		620	1.6505	1669	3.2550	759	4.3827	620	0.7759	759	4.8895	759	0.7844	759	6.36
8	2.41		1568	1.5477	2280	3.1791	2280	4.3377	1778	0.7647	2280	4.6451	1669	0.7599	2280	6.08
9	2.13	2	475	1.4956	475	3.1305	1669	4.3053	1909	0.7629	1669	4.6354	2280	0.7452	1669	6.04
10	1.91		1778	1.4948	620	3.0877	620	4.2142	1265	0.7491	620	4.4902	620	0.7025	620	5.85
11	1.73		1909	1.4759	1909	3.0720	475	4.0850	2280	0.7431	1909	4.3893	1909	0.6805	1909	5.65
12	1.58		1668	1.4232	759	2.9914	1909	4.0829	759	0.7356	475	4.2783	475	0.6745	475	5.55
13	1.45		1445	1.4219	893	2.6948	1445	3.6943	893	0.7088	1445	4.0448	493	0.6456	493	5.18
14	1.34		1265	1.3643	1036	2.6815	493	3.5965	1036	0.7084	2411	3.9948	893	0.6227	1445	5.16
15	1.25		2280	1.3271	1541	2.6576	1668	3.5878	2302	0.6808	493	3.9804	1668	0.6207	2411	5.13
16	1.17		1036	1.3094	493	2.5743	2411	3.5700	1668	0.6550	1282	3.8743	2411	0.6071	1668	5.05
17	1.10		2411	1.3054	1668	2.5557	893	3.5445	1541	0.6537	1668	3.8581	1445	0.6049	1282	4.97
18	1.04		893	1.3047	1445	2.5388	1541	3.5064	1568	0.6507	1541	3.8335	1282	0.6041	893	4.93
19	0.98	1	2302	1.2747	2397	2.5377	1036	3.4932	2397	0.6382	893	3.7717	1541	0.5736	1541	4.90
20	0.93		1432	1.2468	1432	2.4974	1282	3.4269	1432	0.6205	1036	3.7118	1432	0.5710	1036	4.76
21	0.88		1282	1.1749	2411	2.4124	2397	3.3660	1445	0.5860	1432	3.6459	2397	0.5581	1432	4.67
22	0.84		1541	1.1734	2302	2.4053	1432	3.3242	1301	0.5829	2397	3.5734	1036	0.5547	2397	4.63
23	0.81		2397	1.1448	1905	2.3973	1568	3.2750	1905	0.5825	1805	3.5433	1805	0.5515	1805	4.58
24	0.77		347	1.1299	1568	2.3912	1905	3.2489	1249	0.5773	1905	3.4600	2276	0.5464	1905	4.51
25	0.74		1572	1.1249	1805	2.3199	1805	3.2443	493	0.5660	1568	3.4566	1905	0.5434	1568	4.45
26	0.71		493	1.1001	1282	2.3166	2276	3.1055	2283	0.5646	2276	3.3913	347	0.5421	2276	4.42
27	0.68		1301	1.0922	2276	2.2666	347	3.0516	1906	0.5609	347	3.3744	1301	0.5405	347	4.35
28	0.66		1806	1.0872	1301	2.2604	2302	3.0380	2174	0.5584	1301	3.2654	1568	0.5368	1301	4.25
29	0.64		1249	1.0805	2283	2.2213	1301	3.0066	1024	0.5438	2283	3.1946	2283	0.5223	2283	4.18
30	0.61		2283	1.0800	347	2.2182	2283	2.9547	347	0.5386	1917	3.1939	1912	0.5168	1917	4.15
31	0.59		2424	1.0612	1917	2.1184	1917	2.9218	1261	0.5330	2424	3.1638	1806	0.5155	2302	4.04
32	0.58		1906	1.0563	2174	2.0796	2424	2.8232	1805	0.5326	2302	3.1288	1917	0.5085	2424	4.03
33	0.56		1805	1.0558	1261	2.0154	1912	2.8116	2411	0.5301	1912	3.0661	2302	0.5083	1912	4.03
34	0.54		2174	1.0482	1806	2.0125	2174	2.7331	1806	0.5274	1806	3.0210	2424	0.4716	1806	3.97
35	0.53		1905	1.0436	1906	2.0123	1806	2.7097	736	0.5063	607	2.9267	607	0.4702	607	3.75
36	0.51		1024	1.0073	1912	1.9964	607	2.6277	2276	0.5059	2174	2.8695	2174	0.4663	2174	3.74
37	0.50	1/2	900	1.0050	1249	1.9911	1261	2.6246	1917	0.5039	2403	2.7967	711	0.4571	2403	3.60
38	0.48		1917	0.9991	2424	1.9381	613	2.5519	710	0.4973	1261	2.7453	1906	0.4440	613	3.58
39	0.47		607	0.9925	607	1.9294	1906	2.5388	711	0.4933	613	2.7390	613	0.4396	1261	3.55
40	0.46		736	0.9868	736	1.9021	1249	2.5131	900	0.4891	1447	2.6805	1249	0.4322	711	3.51
41	0.45		2403	0.9805	613	1.8873	736	2.4740	1274	0.4871	711	2.6757	2403	0.4276	1906	3.45
42	0.44		1261	0.9710	711	1.8633	2403	2.4683	1282	0.4835	736	2.6559	736	0.4244	1447	3.42
43	0.43		1912	0.9639	1024	1.8361	1447	2.4584	1572	0.4717	1906	2.6266	1261	0.4236	736	3.41
44	0.42		1447	0.9626	1274	1.7733	711	2.4425	607	0.4689	1023	2.6167	1572	0.4213	1249	3.39
45	0.41		710	0.9535	1447	1.7708	1023	2.3465	1268	0.4651	1249	2.6156	1023	0.4137	1023	3.37
46	0.40		2276	0.9418	710	1.7381	2418	2.3080	1912	0.4532	2418	2.5514	1447	0.4129	1572	3.28
47	0.39		711	0.9375	2266	1.7347	1274	2.2988	2424	0.4503	1572	2.4961	1274	0.4072	2418	3.26

Table 6.6 *The Linköping 2 basin. Ranked peak-flow values calculated with a unit hydrograph. Marked groups represent storms with peak-flow values differing from the peak-flow value corresponding to the return period studied by at most ± 5%. Common historical "design storms" are underlined.*

No	Return period year		Rainfall numbers and Peak flow values (m^3/s) for pipe no 7		15		29		35		38		51		54	
1	32.28		596	0.2746	596	0.5174	596	0.8476	596	0.2935	596	0.4218	596	0.9426	596	1.960
2	11.88		2025	0.2015	2025	0.3717	2025	0.7570	2025	0.2146	2025	0.3570	2025	0.7793	2025	1.640
3	7.20		2171	0.1935	2171	0.3578	2171	0.7028	2171	0.2056	2171	0.3335	2171	0.7486	2171	1.572
4	5.16	5	885	0.1734	885	0.3286	885	0.5685	885	0.1914	885	0.2690	885	0.6075	885	1.264
5	4.02		1669	0.1536	475	0.2820	1669	0.4985	1669	0.1599	1669	0.2441	1669	0.5451	1669	1.132
6	3.29		475	0.1472	1669	0.2793	2280	0.4843	475	0.1598	475	0.2423	475	0.5201	475	1.073
7	2.79		2302	0.1395	2280	0.2740	475	0.4750	2280	0.1527	2280	0.2419	620	0.5021	1778	1.042
8	2.41		1036	0.1378	620	0.2562	620	0.4728	1036	0.1520	620	0.2298	1778	0.4949	620	1.037
9	2.13	2	1909	0.1305	1909	0.2545	1778	0.4686	1265	0.1442	1778	0.2290	1909	0.4910	2280	1.028
10	1.91		620	0.1303	1036	0.2516	1909	0.4584	620	0.1440	1909	0.2274	2280	0.4844	1265	1.016
11	1.73		2280	0.1295	1265	0.2488	1265	0.4522	1909	0.1440	1265	0.2241	1265	0.4808	1909	1.014
12	1.58		1568	0.1251	1778	0.2478	759	0.4400	2302	0.1436	759	0.2066	759	0.4741	759	0.984
13	1.45		759	0.1248	2302	0.2372	893	0.4229	1778	0.1423	893	0.2050	893	0.4626	893	0.958
14	1.34		1265	0.1231	893	0.2272	1036	0.4048	1568	0.1358	1036	0.2029	1036	0.4436	1036	0.919
15	1.25		893	0.1226	1568	0.2272	2302	0.3951	893	0.1316	1541	0.1964	2302	0.4340	2302	0.899
16	1.17		1778	0.1215	1541	0.2256	1541	0.3874	759	0.1299	1668	0.1928	1668	0.4168	1668	0.860
17	1.10		1541	0.1165	2397	0.2212	1668	0.3810	2397	0.1285	2397	0.1928	1541	0.4147	1541	0.857
18	1.04	1	2397	0.1156	759	0.2211	2397	0.3793	1541	0.1278	2302	0.1895	1568	0.4077	1568	0.851
19	0.98		1668	0.1143	1668	0.2160	1568	0.3759	1668	0.1258	1568	0.1840	2397	0.4031	2397	0.834
20	0.93		1432	0.1117	1905	0.2059	1432	0.3549	1905	0.1211	1432	0.1797	1432	0.3916	1432	0.818
21	0.88		1024	0.1109	1432	0.2041	493	0.3492	1432	0.1193	1905	0.1779	493	0.3812	493	0.790
22	0.84		1906	0.1076	1249	0.1949	1905	0.3443	493	0.1149	493	0.1768	1301	0.3691	1445	0.764
23	0.81		1249	0.1051	1445	0.1944	1445	0.3439	1249	0.1127	1445	0.1738	1445	0.3670	1301	0.763
24	0.77		1445	0.1045	493	0.1940	1249	0.3376	1024	0.1121	2283	0.1640	2283	0.3657	1905	0.759
25	0.74		1905	0.1040	1906	0.1896	1301	0.3367	1445	0.1121	1906	0.1629	1905	0.3648	2283	0.753
26	0.71		1301	0.1024	1261	0.1889	2283	0.3365	1301	0.1116	2276	0.1626	1249	0.3632	1249	0.745
27	0.68		1261	0.1014	1301	0.1879	2276	0.3303	1261	0.1110	1301	0.1613	1906	0.3620	1906	0.744
28	0.66		2174	0.1003	2174	0.1863	347	0.3242	1906	0.1084	2174	0.1608	2174	0.3546	2174	0.726
29	0.64		900	0.0980	2283	0.1859	1906	0.3196	2174	0.1076	1805	0.1600	1024	0.3545	347	0.722
30	0.61		2283	0.0980	1024	0.1835	2411	0.3184	1805	0.1056	1249	0.1596	347	0.3438	2276	0.722
31	0.59		1806	0.0938	1805	0.1832	1024	0.3168	2283	0.1055	2411	0.1565	1806	0.3413	1024	0.721
32	0.58		347	0.0928	2276	0.1807	2174	0.3160	900	0.1046	1261	0.1529	2411	0.3406	2411	0.717
33	0.56		493	0.0908	347	0.1742	1805	0.3121	2276	0.1030	347	0.1516	2276	0.3404	1805	0.706
34	0.54		1759	0.0906	900	0.1736	1261	0.3070	347	0.1016	1024	0.1484	1261	0.3395	1806	0.703
35	0.53		736	0.0904	1917	0.1705	736	0.3051	1917	0.0998	1806	0.1430	1805	0.3343	1261	0.702
36	0.51		710	0.0903	2411	0.1702	711	0.3021	1806	0.0976	1912	0.1424	710	0.3227	711	0.673
37	0.50	1/2	1805	0.0903	1806	0.1692	1806	0.3014	2411	0.0972	613	0.1404	736	0.3220	1917	0.667
38	0.48		1572	0.0888	1759	0.1671	1917	0.3000	736	0.0962	736	0.1404	711	0.3196	736	0.665
39	0.47		711	0.0875	710	0.1668	1282	0.2893	1759	0.0958	1917	0.1400	1917	0.3196	710	0.660
40	0.46		1268	0.0875	736	0.1656	710	0.2882	1268	0.0947	1274	0.1399	1274	0.3100	1282	0.647
41	0.45		2411	0.0874	1912	0.1647	1274	0.2799	710	0.0946	710	0.1392	1282	0.3097	1274	0.633
42	0.44		1274	0.0861	1023	0.1643	1912	0.2793	1912	0.0946	1282	0.1371	607	0.3004	607	0.625
43	0.43		1912	0.0854	1268	0.1617	607	0.2775	1023	0.0935	711	0.1348	1572	0.2971	1912	0.614
44	0.42		1023	0.0851	613	0.1593	613	0.2775	711	0.0934	2266	0.1292	900	0.2951	613	0.606
45	0.41		2276	0.0851	1572	0.1577	900	0.2747	1282	0.0931	607	0.1288	1912	0.2933	1572	0.604
46	0.40		1539	0.0845	1274	0.1569	1759	0.2668	1572	0.0930	1539	0.1288	613	0.2888	900	0.603
47	0.39		1917	0.0843	619	0.1565	1572	0.2664	613	0.0913	1572	0.1251	1268	0.2874	2424	0.592

7. COMPARISON OF DESIGN USING DESIGN STORMS WITH DESIGN USING HISTORICAL STORMS

7.1 Comparisons Reported in Literature

Comparisons of designs using design storms with design using historical storms have been reported by Marsalek (1977, 1978a, 1978b), Sieker (1978), Wenzel and Vorhees (1978, 1979), Urbonas (1979), and Packman and Kidd (1980).

Marsalek (1978a, 1978b) compared simulated peak-flow values for the Chicago design storm (see Section 3.3) and the Illinois State Water Survey (ISWS) design storm (see Section 3.5) with peak-flow values simulated for historical storms. The runoff simulations were made with the Storm Water Management Model for three catchment sizes (0.26, 0.52, and 1.30 km^2) and with three different values of imperviousness (15, 30, and 45%) in each catchment. Data concerning the design storms were evaluated from the same 15-year rainfall record, from which the historical storms were selected. Marsalek found that the Chicago design storm produced peak flows that were about 75% larger than the peak flows corresponding to the historical storms, and the peak-flow values for the ISWS design storm were about 20% larger than the peak flows for the historical storms. However, the duration of the ISWS storm was fixed to one hour, and a decrease to one-half of an hour increased the peak-flow values by about one-third. The conclusion was that the uncertainty in simulated peak-flow values caused by the choice of design rainfall was larger than the uncertainty caused by other factors involved in the runoff simulations.

Sieker (1978) compared the simulation of peak-flow values with a linear-storage model for historical storms with the peak-flow values for average-intensity-duration rainfalls taken from Intensity-Duration-Frequency curves. The simulations were made for a 0.54 km^2 size runoff area with 38% imperviousness. The result showed that for return periods longer than one year, the historical rainfalls caused larger peak flows than the design storms.

For a return period of 10 years, the difference was about 50% of the flow values for the design storms. His conclusion was that the postulate rainfall frequency equal to peak flow frequency is not correct.

Wenzel and Voorhees (1978, 1979) compared the simulated peak-flow values in two catchments, a large one with a size of 9.20 km^2, and a small one with a size of 0.092 km^2. Simulations were made with a continuous ILLUDAS model for historical storms and two to three design storms. For the large catchment an average-intensity-duration design storm was used, which was obtained from I-D-F curves. These curves were evaluated from the historical rainfalls. The Illinois State Water Survey storm (ISWS) with its original distribution (Huff, 1967) was also used, and in the small catchment a triangular hyetograph was used as well. Dry and wet antecedent-moisture conditions were simulated. The best result was obtained for the ISWS-storm and dry antecedent-moisture conditions. The average-intensity-duration rainfall with dry-antecedent moisture conditions gave an underestimation of peak-flow values. For the large area a duration of 105 minutes was used and for the small area, 15 minutes.

Urbonas (1979) simulated the peak-flow values with a unit hydrograph in four basins for the 73 largest one-hour rainfalls of a 73-year period of record. The basins varied in size from 0.39 km^2 to 1.45 km^2. The resulting peak-flow distribution functions were compared with peak-flow values for local design storms for Denver, Colorado. The design storms overestimated the peak-flow values by 10 to 50% compared with the historical storms. Another result was that antecedent precipitation very little affected the probability distributions of peak-flow values. The work is continuing with the development of a design storm causing peak-flow values matching the values obtained for the historical storms for different frequencies.

Packman and Kidd (1980) and Kidd and Packman (1980) have presented recommendations for the use of design storms published by the Natural Environment Research Council (1975) in the Flood Studies Report (FSR). The FSR-storm input is described by four parameters: rainfall depth, duration, profile peakedness (see Section 3.6), and catchment antecedent wetness index (UCWI). The UCWI value governs the percentage of runoff. In the studies, the rainfall depth was chosen from Intensity-Duration-Frequency curves for the design return period. Different durations were tested in order to find the one which gave the largest peak-flow value. The profile peakedness was fixed to 50%. The studies were therefore limited to giving recommendations for the choice of UCWI, so that the simulated peak flows for the design storm matched the simulated distribution functions for historical storms. The simulations were carried out with a non-linear reservoir surface runoff model, applied to two real catchments and one imaginary catchment plus two imaginary catchments similar to the real ones but located to the wetter part, south-western England. Two series of historical storms were used, one a 98-year record and the other a 34-year record, valid for south-western England. The result showed that the recommendations for the estimation of rainfall input data produced peak-flow values with less than a 10% error compared to peak-flow values simulated for historical storms.

The results of comparisons of simulated peak-flow values for design storms with peak-flow values for historical storms show large variations. If we assume the historical storms give the most correct estimation of design flows, the design storms were found to both overestimate and underestimate the design-flow values. Marsalek (1978a, 1978b) found that the Chicago design storm gives an overdesign. The Illinois State Water Survey storm was also causing too large peak flows, except for dry-antecedent catchment conditions (see Marsalek; and Wenzel and Voorhees, 1978, 1979). Both Wenzel and Voorhees and Sieker (1978) got underestimated flow values for the average-intensity-duration storm. Urbonas (1979) and Packman and Kidd (1980) show that it is possible to develop design storms and rules for the use of them so that the estimated peak-flow values match the peak-flow values caused by historical storms.

Wenzel and Voorhees, and Packman and Kidd report an influence on simulated flow values from antecedent precipitation. Urbonas found no such influence. It seems that local climatic conditions and characteristics in the runoff basins govern the effect of antecedent precipitation.

7.2 Factors Influencing the Comparison of Designs for Different Types of Rainfall Data

For the design points of each test basin used in this study the following statistical analyses were carried out concerning the peak flows.

- The "true" statistical distribution functions were estimated by plotting the peak flows simulated with the CTH-Model for all the historical storms. The results are shown in Appendix III.

- The results of the simulations for different design storms were plotted in the same figures as the results for the historical storms (see Appendix III).

- The results of simulations for historical "design storms" selected by the unit hydrograph method are also plotted in Appendix III.

The interpretation of the result has been made by studying the differences between the design peak flows for different kinds of rainfall data and the design peak flows calculated with the CTH-Model for all historical storms. The differences are listed in Appendix IV (for average values and standard deviations, see Table 7.6). The following factors are also considered in the interpretation.

- The errors and uncertainties inherent in the calculated peak flows due to the fact that the model is not a perfect tool for simulation of the urban runoff process and due to the errors and uncertainties in the model parameter values, especially the rainfall data.

- Changes in investment costs due to over- and underestimations of the peak flows.

- Changes in return periods of the peak flows due to over- and underestimations of the peak-flow values.

Errors and Uncertainties Inherent in the Modeling

A model of the type represented by the CTH-Model is assumed to make simulations of the peak flows within an estimated model error of ± 15% and with a standard deviation of 15-20% (see Section 6.1 and Arnell, 1980). A complete analysis of errors and uncertainties have been made by Yen et al. (1976) and Tung and Mays (1980) while dealing with risk and safety-factors (see Section 1.1) in the design of sewer systems. They have shown it is possible to analyze the errors and uncertainties in the following way.

Substitute the calculation of the peak-flow values with the CTH-Model by the expression

$$Q = \eta \cdot G(x_1, x_2, \ldots, x_j) \qquad \ldots(7.1)$$

where Q = calculated peak-flow value

η = coefficient for compensation of model error where the term model error includes systematic errors caused by the model itself and by the model parameters

$x_1, x_2, \ldots, x_j$ = parameters in the model, such as slopes, roughness, rainfall intensities etc.

The mean value of Q can be estimated as

$$\bar{Q} = \bar{\eta} \cdot G(\bar{x}_1, \bar{x}_2, \ldots, \bar{x}_j) \qquad \ldots(7.2)$$

and the coefficient of variation of Q as

$$\Omega_Q^{\,2} \approx \Omega_\eta^{\,2} + \frac{1}{\bar{G}^2} \sum_j \left(\frac{\partial G}{\partial x_j}\right)^2_{x_j = \bar{x}_j} \cdot \bar{x}_j^2 \cdot \Omega_{x_j}^{\,2} \qquad \ldots(7.3)$$

where Ω_Q = estimated coefficient of variation of Q

Ω_η, Ω_{x_j} = estimated coefficients of variation of η and x_j

The coefficient of variation is the standard deviation divided by the mean value.

The values of η, Ω_η, and Ω_Q for the CTH-Model have been estimated by Arnell (1980) to be 1/0.95, 0.10, and 0.18, respectively, when calibrated sizes of contributing areas were used. The value 0.95 was determined as the average value of the calculated peak-flow values divided by the measured peak-flow values and the value 0.18 is the standard deviation of the same relationship for all rainfall-runoff events used in the validation of the model and that value is assumed to be the estimated value of Ω_Q. The value 0.10 is the standard deviation of the average relationship above for the different catchments used in the validation. If these values are inserted in Eq. (7.3), we obtain

$$0.18^2 \approx 0.10^2 + \underbrace{\frac{1}{\bar{G}^2} \sum \left(\frac{\partial G}{\partial x_j}\right)^2_{x_j = \bar{x}_j} \cdot \bar{x}_j^2 \cdot \Omega_{x_j}^{\,2}}_{A^2} \qquad \ldots(7.4)$$

$A^2 = 0.0224 \quad (A = 0.15)$

Now consider the influence on Ω_Q from the uncertainty in the rainfall intensities, which can be of interest since we are dealing with rainfall data for the design of sewers. Tables 7.1-7.3 show that percentage changes in the rainfall intensity cause about the same percentage changes in the peak-flow values, and thus, the contribution from the rainfall to the coefficient of variation of Q is reduced to $\Omega_i^{\,2}$, or expressed by the last term of Eq. (7.3).

Table 7.1 *The Bergsjön basin. Changes in peak-flow values, number of pipes with a changed diameter, and changes in total costs due to changes in rainfall intensities (i (0%) = 59.0 mm/h in 5 min.).*

Relative rainfall intensity %	Relative peak flow value (%) for design point No. 09	24	30	42	57	66	73	Number of pipes with changed diameter	Changes in total costs 10^3 Sw.cr.	%
+20	+19	+21	+20	+20	+23	+19	+21	8	+16.4	+2.0
+15	+15	+16	+15	+15	+18	+15	+16	8	+16.4	+2.0
+10	+10	+11	+10	+10	+ 9	+ 9	+10	6	+12.8	+1.6
+ 5	+ 5	+ 5	+ 5	+ 5	+ 5	+ 4	+ 6	2	+ 3.5	+0.4
59.0 mm/h	0.103 m^3/s	0.246 m^3/s	0.133 m^3/s	0.108 m^3/s	0.087 m^3/s	0.067 m^3/s	0.612 m^3/s	-	805.8	
- 5	- 5	- 6	- 5	- 6	- 6	- 6	- 5	2	- 6.1	-0.8
-10	-11	-12	-10	-10	-11	-12	-11	4	-12.8	-1.6
-15	-16	-18	-15	-16	-17	-16	-16	7	-26.3	-3.3
-20	-21	-24	-20	-20	-23	-22	-22	11	-41.1	-5.1

Table 7.2 The Linköping 1 basin. Changes in peak-flow values, number of pipes with a changed diameter, and changes in total costs due to changes in rainfall intensities (i (0%) = 41.6 mm/h in 10 min.).

Relative rainfall intensity	Relative peak flow value (%) for design point No.							Number of pipes with changed diameter	Changes in total costs	
%	26	72	73	84	89	106	125		10^3 Sw.cr.	%
+20	+22	+27	+27	+23	+27	+24	+28	49	+1618.1	+15.5
+15	+17	+20	+20	+17	+21	+18	+20	32	+1126.6	+10.8
+10	+11	+12	+12	+12	+12	+12	+12	22	+ 510.0	+ 4.9
+ 5	+ 5	+ 7	+ 7	+ 7	+ 6	+ 6	+ 6	17	+ 326.0	+ 3.1
41.6 mm/h	0.855 m^3/s	1.942 m^3/s	2.684 m^3/s	0.466 m^3/s	2.535 m^3/s	0.504 m^3/s	3.185 m^3/s	-	10442.9	
- 5	- 6	- 6	- 5	- 5	- 6	- 6	- 6	10	- 180.1	- 1.7
-10	-11	-11	-11	-10	-12	-13	-12	24	- 470.7	- 4.5
-15	-16	-17	-17	-15	-19	-19	-19	32	- 637.9	- 6.1
-20	-21	-22	-22	-20	-26	-25	-24	40	-1007.6	- 9.6

Table 7.3 *The Linköping 2 basin. Changes in peak-flow values, number of pipes with a changed diameter, and changes in total costs due to changes in rainfall intensities (i (0%) = 50.5 mm/h in 7 min.).*

Relative rainfall intensity %	Relative peak flow value (%) for design point No. 07	15	29	35	38	51	54	Number of pipes with changed diameter	Changes in total costs 10^3 Sw.cr.	%
+20	+17	+19	+21	+24	+25	+23	+25	9	+ 56.6	+5.2
+15	+13	+16	+14	+19	+19	+17	+19	7	+ 43.9	+4.0
+10	+ 7	+11	+10	+14	+14	+12	+13	6	+ 36.4	+3.3
+ 5	+ 5	+ 4	+ 4	+ 8	+ 8	+ 6	+ 7	5	+ 26.4	+2.4
50.5 mm/h	0.088 m^3/s	0.187 m^3/s	0.323 m^3/s	0.102 m^3/s	0.165 m^3/s	0.340 m^3/s	0.685 m^3/s	-	1098.0	
- 5	- 6	- 5	- 6	- 5	- 5	- 6	- 7	6	- 27.8	-2.5
-10	-11	-11	-11	-11	-12	-11	-12	7	- 29.5	-2.7
-15	-16	-18	-17	-16	-18	-17	-18	9	- 38.0	-3.5
-20	-22	-23	-22	-22	-25	-23	-23	11	- 44.9	-4.1

$$\frac{1}{\bar{G}^2}\left(\frac{\partial G}{\partial x_i}\right)^2_{x_i=\bar{x}_i} \cdot \bar{x}_i^2 \cdot \Omega_i^2 \approx \Omega_i^2 \qquad \ldots(7.5)$$

where i = index for the rainfall parameter

Eq. (7.5) inserted in Eq. (7.4) gives

$$\Omega_Q^2 \approx \Omega_\eta^2 + \underbrace{\Omega_i^2 + B^2 \text{ (remaining parameters)}}_{A^2} \qquad \ldots(7.6)$$

The uncertainties in the different design storms are related to the uncertainties in the Intensity-Duration-Frequency relationships. The Average-Intensity-Duration storms, the Chicago design storms, and the Sifalda design storms are directly connected to the Intensity-Duration-Frequency curves. A change of the curves gives the same change of the important central parts of the design storms. For the Chicago storm an extra uncertainty is introduced through the equations of the I-D-F curves (see Eq. (2.2)). Arnell (1974) has estimated the total error in similar I-D-F curves to be about ± 5-10%. The largest instrument error (excluding the problem of estimating the true precipitation at ground level) is that the vertical axis of the floater is not parallel to the vertical axis of the diagram paper. By studying the emptying of the floating chamber, the magnitude of the error was estimated to be only 2% for the Lundby data for a duration of 5 minutes. This is a very small error, and when the other uncertainties are added, the resulting curves are estimated to be correct with an accuracy of ± 5-10%. It should be pointed out that the historical storms must be marred by the same errors.

The best information we have about the I-D-F curves is the plotted points in Fig. 2.3. From these points the I-D-F curves were estimated by fitting the Log-Pearson type III distribution to the data. The errors thus introduced are approximately 5% for a return period of 1 year and 10% for a return period of 5 years. The intensity values estimated by Eq. (2.2) have standard errors of about 1-2%. So, the Average-Intensity-Duration storms, the Sifalda storms, and the Chicago design storms have uncertainties of $\pm$ 7-14%.

Tables 7.1 - 7.3 show that changes in the rainfall intensities of the Average-Intensity-Duration storms cause about the same or slightly larger changes in the peak-flow values. Thus, the uncertainties in the rainfall intensities above are transformed into uncertainties in the calculated peak-flow values of $\pm$ 7-14%.

Tables 7.1 - 7.3 were obtained through runoff simulations for the Average-Intensity-Duration storms with a return period of one year. The same durations were used for the entire catchments, and the values are given in the headings of the tables. The pipe diameters were designed by the model, starting with 225 mm as the smallest diameter. After the first runoff simulation with rainfall intensities obtained from the I-D-F curves, the intensities were varied within $\pm$ 20%. The variation in peak-flow values and the number of pipes that obtained a changed diameter because of the variation in rainfall intensities are given in Tables 7.1 - 7.3.

For a change in rainfall intensity of $\pm$ 20% the number of pipes that obtained a different diameter was about $\pm$ 30-40% of those with a diameter larger than 225 mm, and similarly for a variation in rainfall intensity of $\pm$ 10% a change of 15-25% was obtained.

If the uncertainties of 7-14% in the rainfall intensities are assumed to be the value of Ω_i, the uncertainty from the remaining parameters can be estimated if we remember that the value of A^2 was 0.0224. If a value of Ω_i of 7% is inserted into Eq. 7.6,

$$
\begin{aligned}
B^2 &= 0.0224 - 0.07^2 \\
B^2 &= 0.0175 \qquad \ldots(7.7) \\
B &= 0.13
\end{aligned}
$$

and for $\Omega_i = 14\%$

$$B^2 = 0.0028 \qquad \text{...(7.8)}$$

$$B = 0.05$$

A value of B = 0.05 seems too low if we consider all the parameters included. I, therefore, assume $B \approx 0.10$ which leads to the estimated changes in Ω_Q due to changes in Ω_i given in Table 7.4.

Table 7.4 *Changes in Ω_Q due to changes in Ω_i.*
$\Omega_Q^2 \approx \Omega_\eta^2 + \Omega_i^2 + B^2$, where $\Omega_\eta \approx 0.10$ and $B \approx 0.10$.

Ω_Q	Ω_i
0.30	0.26
0.25	0.21
0.20	0.14
0.18	0.11

If we accept that Ω_Q increases to 0.25, the value of Ω_i may increase from 0.11 to 0.21. A large value of Ω_i leads to the fact that a larger safety-factor must be used if the same risk of floodings is to be maintained. Therefore, Ω_i should be as small as possible. The analyses above and the figures used should not be interpreted as exact estimates of errors and uncertainties involved but as an indication of the magnitudes of the errors and uncertainties involved.

The systematic differences between the peak-flow values for some of the design storms and the peak flows for the historical storms can be considered as an increase in η. To express it in another way: The combined use of the CTH-Model and a specified design storm gives systematic over- or underestimations that must be compensated for by a coefficient η.

Let us use the I-D-F design storm in the example below. The contribution to η from the design storm was estimated to on the average 1/0.91 (see Table 7.6) and the value of Ω_η to 0.062. This will give a total value of η and an increase in Ω_Q equal to

$$\eta = \frac{1}{0.95} \cdot \frac{1}{0.91} = 1.15 \qquad \ldots(7.9)$$

$$\Omega_Q^2 = 0.18^2 + 0.062^2 = 0.190^2 \qquad \ldots(7.10)$$

Thus, the coefficient of variation for the computed peak flows increases very little due to the over- and under-estimations for the different design storms (except for the FSR design storm). The main effect is the contribution to η. The conclusion from the example and from Table 7.4 is that a standard deviation of 10-15% of the differences between the peak-flow values for the different rainfall data and the peak-flow values for the historical storms can be accepted without a large increase in the total uncertainty.

For practical and logical reasons it must be an advantage to design sewer pipes by a design method in combination with design rainfalls which gives systematic errors as small as possible, or has a value of η as close as possible to unity.

If the data for Lundby 1921-1939 are to be used for design in the future, one must consider that the rainfall data and their statistical characteristics are just an estimate of unknown future rainfall data and their statistics. This uncertainty can be taken into account by estimating confidence limits for the different statistical distribution functions. This is not done here because the data are used for comparison only and all storms come from the same data series. The deviations that should be considered are the deviations in the peak-flow values for the different design storms caused by the characteristics of the rainfall data for Lundby 1921-1939. In other words, would the deviations in peak-flow values for the different design storms have been different if another historical rainfall record had been used? Especially for the longer return periods, the local coefficients of the design storms were estimated from a few historical storms only, which means that the rainfall-parameter mean values are not stable.

Changes in Investment Costs

Coupled with the changes in pipe diameters are changes in investment costs. A rough estimate of the investment costs and the changes in costs due to changes in rainfall intensities is presented in Tables 7.1 - 7.3. An estimate of the costs per meter of pipe for different pipe diameters, given in Table 7.5, was used to calculate the costs. Excavation and backfill costs were obtained from Gustavsberg (1975) and converted by index to costs for January 1, 1981. The fill is used within the building area. Costs for pipe material and laying work were obtained from the local water and sewage works in Göteborg. All costs apply to new building areas. For the reconstruction of old systems, much higher costs may be incurred. The costs in Table 7.5 are uncertain, but the differences in costs of the pipes of different diameters is the interesting information.

Table 7.5 Estimated costs per meter of pipe, including pipe costs, excavation costs, and backfill costs used in this study.

Diameter mm	Costs Sw.Cr.	Diameter mm	Costs Sw.Cr.
225	286	1000	1295
300	332	1200	1804
400	424	1400	2343
500	582	1600	3032
600	649	1800	3585
800	886	2000	3958

The costs for the Bergsjön basin (Table 7.1) show rather small variations when the rainfall intensities vary within ± 20%. The reduction in costs for a reduction in rainfall intensities of -15-20% is about Sw.Cr 25.000-40.000. The variation in costs for the Linköping 1 basin is larger, especially for a variation in rainfall intensities of 10-20%, where the changes in costs are larger than Sw.Cr. 500.000 and as a maximum Sw.Cr. 1.600.000. For the Linköping 2 basin the variation in costs is within the range of Sw.Cr. 25.000-50.000. For the small basins, Bergsjön and Linköping 2, the variations in costs are significant for variations in rainfall intensities of 10-20%. For the Linköping 1 basin the variations in costs are significant also for variations in rainfall intensities of 5%. An overdesign that causes extra investment costs gives a system with less floodings and with future possibilities for connections of new areas so the extra costs can be payed back later.

Changes in Return Periods

An overestimation or underestimation of the peak-flow values in comparison with the values obtained for historical storms leads to changed return periods for the peak flows. The magnitude of the changes can be found in the figures in Appendix III. It can also be estimated if the peak flows in the Appendix are assumed to follow the exponential distribution given by the equation,

$$\ln Q = m + z \cdot \ln F \qquad \ldots(7.11)$$

where Q = peak-flow value

F = return period

m = lower boundary value for the distribution function

z = slope of the distribution function

If the difference is taken between two flow values given by Eq. (7.11) for two different return periods, the following equation is obtained

$$F_2 = \left(\frac{Q_2}{Q_1}\right)^{1/z} \cdot F_1 \qquad \ldots (7.12)$$

where F_1, F_2 = return periods corresponding to peak-flow values Q_1 and Q_2

The variations are rather small in the slopes of the distribution functions for the design points considered, which makes possible a general estimate of the change in return periods due to changes in simulated peak-flow values. The mean value of the slopes for the Bergsjön basin is 0.30; for the Linköping 1 and 2 basins values of 0.37 and 0.34 respectively, were obtained. Thus an overestimation of 10% and 20% of the peak-flow values leads to an increase in the return periods of approximately 30% and 70%, respectively. An underestimation of 10% and 20% of the peak-flow values causes changes in return periods of 27% and 50%. A doubling of the peak-flow value increases the return period 5-10 times.

The figures in Appendix III also show the flow capacities of the pipes with standard diameters, assuming concrete pipes and slopes obtained from the real systems. Because of the rather large differences in capacities between two pipes with standard diameters, a large number of the pipes will eventually have return periods for flooding longer than the one designed for. The exact return period can be determined only by analysis of the whole pipe system at the same time, because the flow values of individual pipes may be influenced by backwater effects from flooded pipes located downstream or reduced flow values from flooded upstream pipes.

Summary of Factors Influencing the Assessment of the Results

In summary, the following factors have been considered when assessing the result of designing pipes for the different types of rainfall data.

- The systematic differences between the peak flows for different storms and the peak flows for all historical storms should be as small as possible.

- The standard deviations of the differences between the peak flows should be as small as possible but due to all the uncertainties involved when simulating the runoff, uncertainties of 10-15% can be accepted without large increases in the total uncertainties.

In addition to these main points, the following two points may be considered.

- For small basins overestimations of simulated peak-flow values of up to 10% can be accepted without involving too large extra investment costs. For large basins even small overestimations cause significant extra investment costs.

- To avoid too large changes in the return periods, the deviations in peak-flow values should be less than 10%.

The judgment of the results of designs for the different types of rainfall data will not only be based on the deviations in peak-flow values in relation to the peak-flow values for all the historical storms. The physical and statistical characteristics of the storms and their development must be included in the assessment as well as their simplicity in practical applications.

7.3 Result of Design Using Design Storms

First, the results of designing for different types of design storms are judged one by one. Then, the results are compared and judged together with results reported in the literature. Underestimations and overestimations of peak-flow values from the design storms were obtained by comparing with the design peak-flow values estimated for the historical storms (see Appendix III and IV and Table 7.6). Comparisons were made for the return periods of 1/2 1, 2, and 5 years. The design peak-flow values for the historical storms were estimated for each return period by linear interpolation between the plotted points (see Table 7.7).

The use of the *Average-Intensity-Duration design storm* (I-D-F storm, see Section 3.2) caused an underestimation of the peak-flow values at most of the design points. The underestimations were smaller in the two smaller basins, Bergsjön and Linköping 2, or on the average about 5-7% (see Table 7.6). For the Linköping 1 basin the underestimations were on the average 14% and as a maximum 22%. For a number of pipes the peak flows were underestimated by more than 10%, which is significant (see Section 7.2).

For some of the design points of the two smaller basins, overestimated peak-flow values were obtained. Generally, the underestimations were larger for the shorter return periods and for the design points close to the outlets.

Table 7.6 *Deviations in simulated peak-flow values for the different design storms as a percentage of the peak-flow values obtained for the historical storms. Mean values (MV) and standard deviations (σ) estimated from values given in Appendix IV for each basin and for all basins together.*

Rainfall data	Bergsjön		Linköping 1		Linköping 2		All basins	
	MV	σ	MV	σ	MV	σ	MV	σ
I-D-F design storm	- 5.5	5.8	-13.9	3.6	- 6.8	5.5	- 8.7	6.2
Chicago design storm	- 2.2	5.5	+10.3	4.4	+ 6.0	3.8	+ 4.7	6.9
Sifalda design storm	- 5.1	4.3	- 0.4	3.7	- 1.7	3.8	- 2.4	4.4
ISWS design storm	- 2.1	6.7	-10.9	5.3	- 3.9	6.4	- 5.6	7.2
FSR design storm	+86.0	30.8	+49.9	24.7	+74.0	27.9	+70.0	31.5
All historical storms and the Unit-Hydrograph Method	+ 0.3	4.1	+ 5.1	7.7	+ 3.0	4.1	+ 2.8	5.9
Selected historical design storms and the CTH-Model	+ 2.0	3.8	+ 1.0	4.7	+ 0.2	2.4	+ 1.1	3.8

The durations of the I-D-F storms that caused the maximum peak-flow values (see Table 6.3) were short, for a few design points as short as 3 minutes. The I-D-F design storm is too simple a design storm. Not only the average intensity is important but also the variation in rainfall intensity within the maximum average intensity period.

The peak-flow values simulated for the *Chicago design storm* (see Section 3.3) are in the Bergsjön basin smaller than the peak-flow values obtained for the historical storms. The underestimations are on the average 2% (see Table 7.6). In the Linköping 1 and Linköping 2 basins overestimations for the Chicago storms by on the average 10% and 6%, respectively, were obtained. For many of the design points, especially in the Linköping 1 basin, the over- and underestimations are larger than 10%. However, the deviations from the values corresponding to the historical storms are surprisingly small for such peaked hyetographs as the Chicago storms.

The results of the simulations of peak-flow values for the *Sifalda design storms* (see Section 3.4) show that the peak-flow values were underestimated in most cases and overestimated in some cases. For the Bergsjön basin the underestimation was 5% on the average (see Table 7.6). Similarly, for the Linköping 1 and Linköping 2 basins average underestimations of 0.4 and 2%, respectively, were obtained. Nearly all peak flows are within ± 10% of the peak flows for the historical storms. The Sifalda storm, which includes the I-D-F storm, gives a larger peak-flow value than the I-D-F storm does, because rainfall is added prior to and after the main part. This causes a "baseflow" before the main burst occurs in the pipe system, and thus a larger peak-flow value is obtained. The durations of the central parts of the Sifalda storms causing the maximum peak flows were in a few cases slightly shorter than the corresponding durations of the I-D-F storms. This can be explained by the generation of the flow from the first part of the rainfall.

When the *Illinois State Water Survey design storm* (ISWS storm, see Section 3.5) was used, underestimated peak-flow values were obtained for most design points. For the smaller Bergsjön and Linköping 2 basins, the underestimations were small or on the average 2% and 4%, respectively. Overestimations were obtained for some design points and for the return periods of 2 and 5 years. For the large Linköping 1 basin the underestimations were on the average 11% and for many design points significantly larger than 10%. The peak-flow values estimated for the ISWS storms are slightly larger than the values estimated for the I-D-F storms due to the more peaked hyetographs of the ISWS storms. However, the peakedness is small and the underestimations of the peak-flow values are significant.

The peak-flow values simulated by means of the *Flood Studies Report design storm* (FSR storm, see Section 3.6) were much larger than the peak-flow values corresponding to the historical storms. The overestimations were more pronounced for the two smaller basins or about 70-85% (see Table 7.6). For the Linköping 1 basin the overestimations were "only" about 50%. The overestimations were larger for the longer return periods of 2 and 5 years, which can be explained by the more peaked hyetographs for these return periods (see Fig. 3.14). The total durations of the FSR storms that caused the maximum peak-flow values were about 3-5 times the corresponding durations for the I-D-F storms. This is in accordance with the recommendations given in Section 3.6.

The large overestimations of peak-flow values obtained for the FSR storms can be explained by the extremely peaked hyetographs of the storms. The large peakedness was obtained when the storms were estimated by centering the historical storms around the most intense parts. The resulting storms are *mean* values of the volume accumulated symmetrically around the center. It may have been more in accordance with the original evaluation reported by the Natural Environment Research Council (1975) if the *median* values had been used. A check based on the 5-year return period storm gave, however, an even more peaked storm for the median values than for the mean values. The evaluations were made for the most intense parts of the storms with a total duration of 240 minutes. No significant differences for different total durations were reported by the National Environment Research Council, so the influence from the choice of the duration of 240 minutes on the resulting profiles has not been investigated. A satisfactory explanation for the extremely peaked hyetograph compared to the English one cannot be given. Nevertheless, the results of the use of the local FSR-design storm are presented.

7.4 Result of Design Using Historical Storms

Design with the CTH-Model Using All Historical Storms

The results of the designs using the historical storms are given in Appendix III as statistical distribution functions for the different design points. No mathematical distribution functions were fitted to the plotted points because the points themselves are the best information we have. The design flows for the studied return periods, i.e. 1/2, 1, 2, and 5 years, were estimated by a linear interpolation between the plotted points, and the results are listed in Table 7.7. Since the plotted points close to the return periods of 1/2, 1, and 2 years are average values of three flow values, the method includes a kind of smoothing of the distribution functions. These flow values are assumed to be the most correct values available. Of course, if measured peak-flow values had been available for all design points and for the rainfalls used in the runoff simulations, these values would have been the best. In that case it would have been possible to study the differences between the measured design flows and the design flows estimated with the CTH-Model.

Arnell (1978) made a comparison between the distribution functions of measured peak flows and calculated peak

Table 7.7 Design peak-flow values for the Bergsjön, the Linköping 1, and the Linköping 2 basins estimated with the CTH-Model for historical storms from Lundby, Göteborg, 1921-1939.

Catch-ment	Design point No.	Design flow value (m^3/s) for return period (year) 1/2	1	2	5
Bergsjön	09	0.103	0.124	0.150	0.195
	24	0.204	0.260	0.306	0.394
	30	0.140	0.165	0.200	0.273
	42	0.094	0.116	0.132	0.172
	57	0.070	0.093	0.108	0.138
	66	0.058	0.074	0.086	0.111
	73	0.573	0.696	0.848	1.134
Linköping 1	26	1.010	1.260	1.480	2.010
	72	1.770	2.330	2.960	3.650
	73	2.400	3.250	4.100	5.350
	84	0.490	0.630	0.770	0.985
	89	2.420	3.240	4.100	5.750
	106	0.455	0.615	0.770	0.990
	125	3.040	4.100	5.200	7.300
Linköping 2	07	0.090	0.115	0.133	0.166
	15	0.155	0.211	0.253	0.316
	29	0.267	0.347	0.450	0.535
	35	0.088	0.120	0.145	0.182
	38	0.138	0.180	0.230	0.284
	51	0.310	0.388	0.490	0.595
	54	0.625	0.790	1.015	1.220

flows for two years (1973-1974) of data for the Bergsjön basin. No calibration of sizes of contributing areas and surface depression storage was made. The result showed that the model overestimated the peak-flow values by about 20%. Most of that difference could be explained by the difficulties in determining the areas supporting the runoff, and thus the differences did not affect the comparisons of designs for different types of rainfall data.

The most correct design method is to estimate the whole, or a part, of the complete statistical peak flow distribution functions for all pipes to be designed in a sewer system. This is a time-consuming task, but the work can be facilitated if a computer is used to store the peak flows, to automatically make the statistical analyses, and to estimate the design peak-flow values (and the necessary standard pipe diameters) corresponding to the design return period. This is also a probable development when computers become faster and cheaper, but today the computer costs are high when using a runoff model like the CTH-Model.

The method used for the rough screening of the rainfall record to select the heaviest rainfalls for runoff simulations was found good enough. The selection was made (see Section 6.4) by identification of the 54 largest storms in ranked lists of maximum average intensities for durations of from 5 minutes to 30 minutes (Linköping 1, 120 minutes). The aim was to obtain enough data for a correct statistical estimate of peak flows for a return period of 1/2 year and longer.

Design with the Unit-Hydrograph Method Using All Historical Storms.

To avoid the high costs, a simpler model coupled with all historical storms can be used in the final design. Since the unit-hydrograph method has been tested for the selection of historical design storms (see Section 6.5), one can also investigate which design flows would be obtained if the unit hydrograph method was used in the final design. Table 7.8 lists the differences between the design peak-flow values estimated by the unit-hydrograph method compared with the values estimated with the CTH-Model and given in Table 7.7. The unit hydrographs were estimated for a constant rainfall intensity of 41.6 mm/h preceded by an intensity of 4.16 mm/h for the Bergsjön and the Linköping 2 basins, and 28.7 mm/h and 2.87 mm/h, respectively, for the Linköping 1 basin (see Section 6.5). As can be seen in Table 7.8, most of the peak flows calculated by the unit-hydrograph method are close to the

Table 7.8 Differences between the design peak-flow values estimated by the unit-hydrograph method and the values estimated with the CTH-Model. Rainfall intensities for the estimation of the unit hydrographs equal to 41.6 mm/h preceded by 4.16 mm/h for the Bergsjön and the Linköping 2 basins, and 28.7 mm/h and 2.87 mm/h respectively for the Linköping 1 basin.

Catchment	Design point No.	Differences in peak flow values (%) for return period (year)			
		1/2	1	2	5
Bergsjön	09	+ 5	+ 7	+ 5	+ 2
	24	- 2	+ 4	- 1	- 5
	30	- 1	+ 3	- 2	-12
	42	+ 2	+ 4	+ 1	+ 3
	57	+ 2	+ 1	- 1	- 5
	66	0	- 2	- 2	- 5
	73	+ 7	+ 2	+ 1	- 2
Linköping 1	26	- 1	+ 2	+ 1	- 8
	72	+13	+ 9	+ 5	+ 9
	73	+ 9	+ 8	+ 4	- 1
	84	+ 3	+ 2	- 2	- 5
	89	+16	+17	+11	+ 5
	106	+ 1	- 5	- 7	- 3
	125	+18	+20	+14	+ 7
Linköping 2	07	0	0	- 2	+ 3
	15	+ 9	+ 3	0	+ 2
	29	+13	+ 9	+ 3	+ 4
	35	+11	+ 5	- 1	+ 3
	38	+ 2	+ 3	- 1	- 7
	51	+ 4	+ 4	- 1	+ 1
	54	+ 7	+ 6	+ 1	+ 2

values calculated with the CTH-Model, especially for the two smaller basins, Bergsjön and Linköping 2. For the larger Linköping 1 basin, the deviations are 10-20% for the design points 89 and 125. Through these points the largest parts of the basin are drained.

In Table 7.6 the result of the design based on the unit hydrograph is compared with the design using the different design storms. The design based on the unit hydrograph is on the average slightly better than the design for all the other design storms but the Sifalda storm.

The unit hydrographs for the different design points were all estimated for the same constant rainfall intensity. In reality the unit hydrographs should be estimated for different rainfall intensities for the different design points because the time of concentration varies for the points. The constant rainfall intensity should also be related to the intensity of the historical storms and thus be different for different return periods and different historical storms. The influence on the resulting S-hydrographs of different constant rainfall intensities has been shown earlier in reports by Arnell et al. (1980) and by Lyngfelt (1981).

In order to get an indication of the influence on computed peak flows if the rainfall intensity for the generation of the unit hydrographs is changed, new unit hydrographs were generated with the CTH-Model for the design points of the Linköping 1 basin for a constant intensity of 14.35 mm/h, preceded by an intensity of 1.43 mm/h, which is half of the intensities used previously.

The results of using the new unit hydrographs in the final design and estimating the complete statistical distribution functions were investigated. The results are given in Table 7.9 as deviations from the values estimated with the CTH-Model and should be compared with the values given in Table 7.8. The design peak-flow values have decreased 10-15% for all design points. Most of the design

Table 7.9 *Differences between the design peak-flow values estimated by the unit-hydrograph method and the values estimated with the CTH-Model, Linköping 1 basin. Rainfall intensity for the estimation of the unit hydrographs equal to 14.35 mm/h preceded by 1.43 mm/h.*

Catchment	Design point No.	Differences in peak-flow values (%) for return period (year) 1/2	1	2	5
Linköping 1	26	-11	- 8	- 8	-20
	72	- 3	- 3	- 8	- 0
	73	- 5	- 6	-11	- 6
	84	- 8	- 9	- 8	-14
	89	+ 0	+ 2	- 5	- 5
	106	-15	-16	-19	-10
	125	+ 2	+ 4	- 3	- 2

peak flows are now underestimated compared to the values obtained by the CTH-Model. For a few design points, especially 89 and 125, the differences in peak-flow values estimated by the two methods are small, and the agreement is better for this lower unit hydrograph rainfall intensity than for the higher one. This is logical because the times of concentration for the design points 89 and 125 are longer than for the other points.

A possible development of the method of using the unit hydrographs is to include the use of different unit hydrographs for different rainfalls and to study which constant intensity to use for the generation of the unit hydrograph in relation to the intensity of the historical storms. Such a development should give a unit-hydrograph method capable of making designs similar to those made with a more detailed model as the CTH-Model. However, this requires a number of unit hydrographs for each pipe of the sewer system, and thus a large number of unit hydrographs must be generated and stored in the computer. Johansen (1981) has used that technique when estimating overflow volumes in combined sewer systems. To make the final design for all pipes in the system with all the unit hydrographs is about as costly as making the design directly with the CTH-Model, at least with the computer (IBM 3033N) used in this investigation. The large savings appear when the interest is focused on a few pipes of the system only.

Design with the CTH-Model Using Historical "Design" Storms Selected by the Unit-Hydrograph Method.

The results of the designs made with the CTH-Model for the historical rainfalls selected by the unit-hydrograph method are given in Appendixes III and IV and in Table 7.6.

The use of the unit hydrograph for the selection of the historical design storms has given a good result. The average deviation in relation to the peak flows estimated for all historical storms is about 1% (see Table 7.6), and the largest positive and negative deviations listed in Appendix IV are + 10% and - 13%, respectively. Extreme deviations do not exist.

The underlying assumption of the method suggested by Johansen (1979) is that the rainfalls corresponding to the peak-flow values are ranked in the same order whether the peak flows are calculated with a unit hydrograph or with a more detailed model. In order to test that assumption, the Spearman rank-correlation coefficients (Siegel, 1956) were calculated between the ranks of the peak-flow values estimated by the unit-hydrograph method and with the CTH-Model (see Table 7.10). This was done for the 74 largest peak flows of the Bergsjön and Linköping 2 basins and for the 94 largest peak flows of the Linköping 1 basin. In the table average values are given of the peak-flow values calculated by the unit-hydrograph method and with the CTH-Model.

The peak-flow values calculated by the two methods are very similar for the Bergsjön basin. For the Linköping 1 and Linköping 2 basins, the unit-hydrograph method over-

Table 7.10 Estimated values of the Spearman rank correlation coefficient of the correlation between the ranks of the peak-flow values calculated by the unit-hydrograph method and with the CTH-Model, and average values of the peak flows calculated by the two methods. Rainfall intensities for the estimation of the unit hydrographs equal to 41.6 mm/h preceded by 4.16 mm/h for the Bergsjön and the Linköping 2 basins, and 28.7 mm/h and 2.87 mm/h respectively for the Linköping 1 basin.

Catchment	Design Point No.	Spearman correlation coefficient	$\bar{Q}$ Unit hydrograph m^3/s	$\bar{Q}$ CTH-Model m^3/s
Bergsjön	09	0.98	0.113	0.106
	24	0.98	0.221	0.221
	30	0.99	0.144	0.145
	42	0.98	0.102	0.100
	57	0.99	0.078	0.078
	66	0.99	0.062	0.063
	73	0.98	0.633	0.613
Linköping 1	26	0.99	0.967	0.949
	72	0.97	1.916	1.744
	73	0.97	2.612	2.435
	84	0.99	0.476	0.469
	89	0.94	2.845	2.427
	106	0.98	0.453	0.455
	125	0.93	3.685	3.068
Linköping 2	07	0.98	0.098	0.096
	15	0.96	0.184	0.175
	29	0.97	0.324	0.302
	35	0.95	0.107	0.100
	38	0.97	0.158	0.155
	51	0.98	0.349	0.340
	54	0.97	0.725	0.686

estimates the peak flows, which is also shown in Table 7.8. The values of the Spearman rank correlation coefficients vary between 0.93 and 0.99, indicating a high correlation between the two ranks of the peak-flow values.

To make possible a better assessment of the assumption behind the selection of storms by the unit-hydrograph method, we analyzed the differences between the peak-flow values estimated with the CTH-Model for the historical rainfalls belonging to the $\pm$ 5% group that were selected by the unit-hydrograph method and the design peak-flow values listed in Table 7.7. The result is given in Table 7.11 as the mean value (MV) and the value of the standard deviation (σ) for the storms connected with each design point and for each return period. The values in Table 7.11 can be interpreted as a measure of the over- and underestimations that could have been obtained for the different design points and return periods (the 5-year return period is excluded because there is only one storm in each group). The figures must be judged with care since they are calculated from a small number of storms for each group (between 4 and 14, see Tables 6.4 - 6.6). A study of all deviations for a return period of one year for the Linköping 1 basin showed that the deviations are normally distributed, which means that about one third of the differences are outside the range of MV $\pm$ σ.

For the Bergsjön basin the variations described by MV $\pm$ σ (the mean value plus/minus the standard deviation) are within 5 - 15%, which is within acceptable limits. For the Linköping 1 basin the same variations are 5 - 30%. The variation is the largest for a return period of 1/2 year, for which there is a risk of significant underestimations. For a return period of two years the variation is within 10%. The variations for the Linköping 2 basin are 5 - 15%, which is also acceptable. The largest underestimations appear for rainfalls including high rainfall intensities of short durations. The unit-hydrograph method overestimates the peak-flow values for such rainfalls because it is a linear method. When the CTH-Model is used, such rain-

Table 7.11 Deviations (%) between the peak-flow values calculated with the CTH-Model for the historical rainfalls belonging to the ± 5% group selected by the unit-hydrograph method, and the design peak flows estimated with the CTH-Model for all historical storms (see Table 7.7). The table shows the mean values (MV) and the standard deviations (σ) for the storms belonging to each group. Figures within parentheses belong to ± 2.5% groups. Rainfall intensities for the estimation of the unit hydrographs equal to 41.6 mm/h preceded by 4.16 mm/h for the Bergsjön and the Linköping 2 basins, and 28.7 mm/h and 2.87 mm/h respectively for the Linköping 1 basin.

Catchment	Design point	Mean value (MV) and standard deviation (σ) of the differences (%) for each return period (year).					
		1/2		1		2	
	No.	MV	σ	MV	σ	MV	σ
Bergsjön	09	(- 1.6) - 0.7	(3.8) 4.1	- 0.4	5.0	(- 1.7) 0.1	(4.5) 6.3
	24	(- 4.3) - 3.9	(6.5) 8.1	(2.5) 4.2	(1.8) 4.1	- 0.9	5.5
	30	- 2.8	4.2	3.0	5.9	- 0.1	2.8
	42	(- 0.7) 2.9	(5.4) 9.0	3.3	7.1	- 0.7	5.6
	57	1.0	5.3	3.4	1.7	- 1.2	6.6
	66	- 1.1	5.7	- 1.0	3.1	0.3	1.9
	73	5.8	2.3	- 4.3	7.4	- 0.6	4.1
All pipes		- 0.4	6.5	0.6	6.2	- 0.4	4.7
Linköping	26	- 5.1	4.9	1.6	3.7	- 1.9	5.5
	72	- 3.2	10.4	2.3	6.9	1.2	7.5
	73	- 6.7	14.1	(0.5) 0.5	(10.2) 9.9	1.6	11.0
	84	0.5	6.3	2.9	5.1	0.7	4.0
	89	- 8.9	16.9	(1.2) 1.9	(14.9) 12.2	- 0.4	6.8
	106	- 3.4	10.5	- 5.1	9.1	(1.5	6.6)
	125	(-15.2) -11.9	(19.0) 18.5	4.9	14.9	- 0.8	6.8
All pipes		- 5.4	12.0	1.2	9.6	0.2	6.7
Linköping	07	- 1.5	5.3	- 2.8	6.5	- 1.7	3.4
	15	(- 3.1) - 3.3	(13.7) 11.3	6.1	4.0	- 0.5	6.2
	29	4.1	9.4	1.1	3.7	0.6	3.3
	35	(- 4.6) - 4.3	(18.4) 13.5	4.0	9.7	- 1.4	8.7
	38	- 1.1	7.2	0.4	6.7	2.8	3.1
	51	0.3	5.1	2.1	5.6	1.2	3.8
	54	- 1.6	6.9	1.7	5.9	0.5	2.7
All pipes		- 1.4	9.3	1.9	6.4	0.2	4.6
All basins/ All pipes		- 2.1	9.5	1.2	7.7	- 0.0	5.3

fall peaks are attenuated in the runoff process. It is reasonable that larger variations are obtained for the larger Linköping 1 basin than for the smaller basins because the linear unit-hydrograph method should be more applicable to small basins than to large ones. The lowest values of the Spearman rank-correlation coefficients in Table 7.10 appear for the same pipes and return periods as do the largest deviations in Table 7.11. Although the values of the Spearman coefficients are generally high, the small variations indicate differences between the ranking of the peak flows calculated by the unit-hydrograph method and the ranking of the peak flows calculated with the CTH-Model.

The values of MV and σ given in Table 7.11 are larger for the shorter return periods than for the longer ones, which is reasonable, since the consonance between the two ranking lists is better for the longer return periods. A possible development of the method could be to use different values of the allowed deviations for different return periods when selecting the group of representative storms by the unit-hydrograph method.

In order to test the effect of using other limits, the limits of $\pm$ 2.5% were tested for a few design points and return periods. The results are the figures within parentheses in Table 7.11. For the Bergsjön basin the variations decrease, but for the Linköping 1 and Linköping 2 basins the effects are negligible. This must be due to the fact that the variations not only are a result of the limits $\pm$ 5% or $\pm$ 2.5% but are mainly an effect of the different order of ranking of the rainfalls for the unit-hydrograph method and for the CTH-Model.

When the group of representative storms is reduced, the number of storms identical for several design points is reduced, and the final design must be carried out for a larger number of historical design storms; that is, however, a small disadvantage only. The discussion above has been limited to the design points, and one can expect slightly larger deviations (between peak flows estimated for the historical design storms and the peak flows estimated for all historical storms) for the pipes located between the design points because the estimated unit hydrographs and the selected historical design storms are valid for the design points only, and the same historical storm is used for the calculation of the design peak flows for the pipes surrounding a design point as for the design point itself.

As was stated earlier, the resulting flow values of the runoff simulations by the unit-hydrograph method depend on the magnitude of the rainfall intensity used for the estimation of the S-hydrographs. In order to test if the ranking of the peak-flow values also depends on the rainfall intensity, the runoff simulations for the Linköping 1 basin by the unit-hydrograph method were repeated with a unit hydrograph estimated for a rainfall intensity of 14.35 mm/h, preceded by an intensity of 1.43 mm/h. These intensities are half of the values used in the previous investigations.

The values of the Spearman rank-correlation coefficients given in Table 7.12 increased compared to the values given in Table 7.10, especially for the design points 89 and 125. This indicates a better consonance between the ranking of the peak flows estimated by the unit-hydrograph method and those obtained with the CTH-Model. The mean values and the standard deviations of the differences between the peak-flow values estimated with the CTH-Model for the rainfalls belonging to each $\pm$ 5% group and the design flow values given in Table 7.7 have decreased slightly for the design points 89 and 125 (see Table 7.13 and compare with Table 7.11), but still the variations expressed through MV $\pm$ σ are 5 - 22%.

Table 7.12 *Estimated values of the Spearman rank-correlation coefficient of the correlation between the ranks of the peak-flow values calculated by the unit-hydrograph method and with the CTH-Model, and average values of the peak flows calculated by the two methods. Linköping 1 basin. Rainfall intensity for the estimation of the unit hydrographs equal to 14.35 mm/h preceded by 1.43 mm/h.*

Catchment	Design point No.	Spearman correlation coefficient	$\overline{Q}$ Unit hydrograph	$\overline{Q}$ CTH-Model
Linköping 1	26	0.99	0.871	0.949
	72	0.98	1.705	1.744
	73	0.98	2.307	2.435
	84	0.99	0.430	0.469
	89	0.97	2.505	2.427
	106	0.99	0.397	0.455
	125	0.96	3.246	3.068

Table 7.13 *Deviations (%) between the peak-flow values calculated with the CTH-Model for the historical rainfalls belonging to the ± 5% group selected by the unit-hydrograph method, and the design peak flows estimated with the CTH-Model for all historical storms. The table shows the mean values (MV) and the standard deviations (σ) for the storms belonging to each group. Linköping 1 basin. Rainfall intensity for the estimation of the unit hydrographs equal to 14.35 mm/h preceded by 1.43 mm/h.*

Catchment	Design point	Mean value (MV) and standard deviation (σ) of the differences (%) for each return period (year).					
		1/2		1		2	
	No.	MV	σ	MV	σ	MV	σ
Linköping 1	26	- 4.0	6.9	1.8	3.5	- 0.4	4.7
	72	- 5.1	10.3	1.7	6.4	1.2	7.5
	73	- 0.9	10.3	- 0.7	8.8	1.9	6.1
	84	1.5	5.8	0.7	5.9	1.6	4.9
	89	- 9.8	12.7	1.8	13.0	- 1.7	5.8
	106	- 7.6	6.0	- 5.7	10.7	1.5	6.6
	125	- 9.5	13.1	- 1.2	10.5	- 0.8	6.8
All pipes		- 4.6	9.6	0.3	7.5	0.4	5.6

In summary, when the rainfall intensity for the estimation of the unit hydrographs was changed, the results of the designs were better for some of the design points, but still the risk for especially significant underestimations exists. The results can be further improved if a number of unit hydrographs are estimated for different rainfall intensities for each design point. When the runoff simulation is carried out for a specific historical storm, the unit hydrograph related to the characteristic intensity of the storm is used. As the characteristic rainfall intensity of the storm, probably the maximum average intensity for a duration equal to the time of concentration for the considered design point can be used. All these improvements require a special investigation.

7.5 Discussion of the Results and Conclusions

The results are summarized in Table 7.14.

Design Storms

Among all the design storms only the FSR-storm causes peak-flow values that have significant deviations from the peak-flow values obtained for the historical storms. The underestimations for the I-D-F storms are rather large in some cases and can be explained by the fact that it is a very simple storm, a constant rainfall intensity only for the whole duration. Rainfall prior to and after that duration is ignored. Also Sieker (1978), and Wenzel and Voorhees (1978, 1979) got underestimated peak-flow values for the I-D-F storms. Because of the oversimplified hyetograph, the systematic underestimation, and similar results found by other researchers, the I-D-F design storm is not recommended for use in combination with detailed numerical runoff models for the design of sewer pipes.

The Chicago design storm caused in most cases a small overestimation, but the peak flow is still not unacceptable compared to the uncertainties discussed in Chapter 7.2. Marsalek (1978a, 1978b) found that the Chicago de-

Table 7.14 Summary of the comparison of design of sewer pipes for different types of rainfall data.

Rainfall	Average value of percentage deviations	Standard deviation	Characteristics concerning physical background and fitness for use
I-D-F design storm	- 8.7	6.2	Constant intensity during the storm duration. Too small volume. Easy to use and local data available at all places in Sweden.
Chicago design storm	+ 4.7	6.9	Unrealistically peaked storm. Return period longer than the one assigned. Includes all durations. Very easy to use. Some local data available at all places.
Sifalda design storm	- 2.4	4.4	Constant intensity during the central part of the storm. Correct total volume. Easy to use. Some local data available at all places.
ISWS design storm	- 5.6	7.2	Average time-varying rainfall intensity during the storm duration. Too small volume. Easy to use. Only total volumes available at all places.
FSR design storm	+70.0	31.5	Unrealistically peaked hyetograph, which comprises the centered average time-varying rainfall intensity. Easy to use. Only total volumes available at all places.
All historical storms and the CTH-Model	-	-	The most exact method if the model is calibrated. Laborious and expensive to use. Local data available at a few places only.
All historical storms and the Unit-Hydrograph Method	+ 2.8	5.9	The simulated flow values may be marred by smaller or larger errors, especially for single rainfalls. Practical to use if only a few design points are to be analyzed. Expensive to use if many design points are to be studied. Local data available at a few places only.
Selected historical design storms and the CTH-Model	+ 1.1	3.8	The method has given a good result, but the risk for especially underestimations is significant. Difficult to select the rainfall intensity for the generation of the unit hydrographs. Laborious to use. Local data available at a few places only.

sign storm gives a large overdesign, which also can be expected due to the method used for its development from the I-D-F curve (see Section 3.3). Since different points of an I-D-F curve may come from different historical rainfalls, the return period of the Chicago design storm should be longer than the return period for the indivudual points of the corresponding I-D-F curve. This longer return period should lead to an overestimation of the peak-flow values. Therefore, the use of the Chicago design storm is not recommended for sewer pipe design.

Nearly all calculated peak-flow values for the Sifalda design storms are located within "acceptable" deviations from the peak-flow values corresponding to the historical storms. On the average it causes small underestimations. It has a more correct total volume than the I-D-F storm has due to the rainfall added prior to and after the main part (part 2). This rainfall can probably be used in the design of sewer pipe systems. Further studies need to be carried out concerning the influence on simulated peak-flow values of the total duration of the storm, which is governed by the length of the rain-free period between independent historical rainfalls. This, in turn, governs the total volume of the rainfall, and thus the volumes of part 1 and 3 of the Sifalda design storm, which may be important if the storm is used for the design of retention basins.

The local ISWS storm is very similar to the I-D-F storm. The hyetograph is a bit more peaked, but not enough to not give significant underestimations. Since there are better design storms, the use of the local ISWS design storm is not recommended. It is possible that local ISWS design storms for other places may give a better design but that must be checked at each place.

The large overestimations obtained for the FSR-storms cannot be fully explained here. Packman and Kidd (1980) obtained good results for the English version of the storm when antecedent wetness was taken into consideration also. Further work is needed before conclusions can be drawn concerning the FSR storms.

Historical Storms

The use of all the heaviest historical rainfalls was in this report assumed to give the most correct result in combination with the CTH-Model or another good, well-tested non-linear runoff model, and this is also assumed to be the best method for sewer-pipe design.

Rather good results were obtained when using all the heaviest historical storms in combination with the simple unit-hydrograph method for the final design. This method may give large errors for single rainfalls, but the statistical distribution functions, and thus the design peak-flow values, are not so much affected. Further studies are needed concerning the influence on the calculated peak-flow values due to the rain intensity for which the unit hydrographs are estimated combined with some characteristic intensity of the historical storms. After these studies the method may be better suited for the design of sewer-pipe systems. The method is as expensive to run as a model like the CTH-Model, but it has the advantage of being possible to run on a small computer for which the costs for simulations during long time periods are negligible. The technique using the unit-hydrograph method for the final design is especially suitable when the interest is focused on a few pipes of the system only.

The technique of selection of historical design storms with the aid of the unit-hydrograph method has given good results for the examples given in this report. However, the risk for especially significant understimations is not negligible. It seems as if the risk is an effect of the unit-hydrograph method being a linear model. A change in the rainfall intensity for the generation of the unit hydrograph changes the peak-flow values, but the ranking of the values will still not be the same as that obtained by a nonlinear model.

It is possible that the rankings will better coincide if a number of unit hydrographs estimated for different constant rainfall intensities are used at each design point. When the runoff calculations are made for a historical storm, the appropriate unit hydrograph is used. This requires the development of a number of computer programs to handle the unit hydrographs and the runoff simulations for these if the method is still to be called "a simple method for the selection of historical design storms."

The risk for over- and underestimations of the design peak flows calculated for the selected historical storms is difficult to compare with the result of using design storms because the use of design storms has not been analysed in the similar way.

Conclusions

The examples given in this report have not shown that significantly better results are obtained in the design when historical storms are used instead of design storms. Of course, if a good non-linear runoff model is used together with a sufficient number of historical storms, the result will be the best we can obtain. This will also be valid for runoff areas of all sizes and shapes as long as there are no surcharging or backwater effects in the pipe system. The examples given in this report show that it is possible to obtain satisfactory results also for designs with design storms for runoff areas that are not very large and with normal structures of the sewer systems. However, it is difficult to give the results of designs with design storms for very large runoff basins and basins with an abnormal structure of the sewer system, e.g. like an hourglass.

Because of the results obtained in this report and because of the advantages of using design storms, design storms can be used as long as the work is limited to normal runoff areas of sizes up to a few hundred hectares and as long as the engineer is conscious of the limitations of using design storms.

The conclusions concerning rainfall data for the design of sewer-pipe systems are:

- The use of the Sifalda design storms gave the best results for the examples of designs using design storms shown in this report.
- The use of the Average-Intensity-Duration design storms and the Illinois State Water Survey design storms gave underestimated peak-flow values for the examples. For this reason and because they consist of more or less a constant intensity only, they are not recommended for use.
- The use of the Chicago design storm gave small overestimations for the examples shown. Because of its agreement with the complete I-D-F curve and results found by other researchers, smaller or larger overestimations can be expected and therefore, the Chicago design storm is not recommended for use.
- Further studies are needed before conclusions can be drawn concerning the Flood Studies Report design storm.
- Acceptable results of designs by the unit-hydrograph method can be obtained if a correct rainfall intensity is used for the estimation of the unit hydrograph for each design point and for the return period studied.
- Good results of designs were obtained in the examples for historical design storm selected by the unit-hydrograph method. However, the risk for especially significant underestimations is obvious, and further studies of the rainfall intensities used for the estimation of the unit hydrographs for each design

points and coupled with the characteristic intensity of the individual historical rainfalls are needed. It is difficult to make a just comparison between the result of using selected historical design storms and the result of using design storms because the analyses are not similar in all parts.

- The examples given in this report do not show that the use of design storms gives significantly worse designs than the use of historical storms as long as the interest is focused on the peak flows only.

It must be stressed that the results are valid for the design of sewer pipes only and that no surcharging or backwater effects are permitted. The results cannot be extended to the design of retention basins, overflow constructions, or pipes downstream of these devices.

8. RECOMMENDATIONS FOR PRACTICAL APPLICATIONS AND FURTHER RESEARCH

8.1 Recommendations for Practical Applications

Access to rainfall data in the form of continuous time series of rain-intensity values is necessary in order to select the best rainfall data for each practical application. If no time series is available, it is necessary to start measurements of rain intensities with good resolutions in time (~1 minute) and rain volume (~0.1 mm).

If one or several time series are available, but not treated for urban hydrology use, or not stored in a computer memory, it is recommended that the time series be transferred to a computer. After that, major rainfall events can be identified and used for runoff calculations, or the continuous time series can be applied directly if a continuous runoff model is used. Local coefficients for different design storms can be estimated as well as different statistical information concerning the rainfall data.

The best design is obtained when data of all heavy historical rainfalls are used combined with a good runoff model. This method is recommended when accurate calculations of peak flows are desired and when detailed information on the runoff basins is available. Examples of applications where this method is suitable are the design and analysis of pipe-systems in existing areas for which detailed mapping has been made earlier. Other applications where it can be worthwhile to use historical rainfall data are the connection of new drainage areas to existing sewer systems for which the unoccupied flow capacities must be determined and the design of the pipe systems in large areas for which the investment costs are high.

After careful studies of the unit-hydrograph method, especially the connection between the intensity of the individual events and the intensity for which the unit hydrograph is estimated, it may be possible to use it for design combined with all heavy historical storms. The unit-hydrograph method will not give as accurate flow values as a method like the CTH-Model, but if the information on the runoff basin is limited, it can give runoff values that are good enough. In some cases it is a greater advantage to be able to estimate the complete statistical distribution functions than to simulate the exact response for each historical storm. This is the case if flow values for a number of return periods are desired.

When historical storms are used, the interesting rainfalls for use in the runoff simulations can be selected from ranked lists of the maximum average intensities for different durations. It seems as if the duration to use corresponds to the time of concentration, but for some design points the duration may be shorter, so a safety margin should be included.

Runoff calculations for design storms can be made for normal runoff areas of sizes up to a couple of hundred hectares. The best results in the examples were obtained for the local Sifalda design storm. If possible, local coefficients should be evaluated before the storm is used at a new place. Different durations of part 2 must be applied to find the one that gives the maximum peak-flow value. For pipes far upstream in a basin, durations of only a few minutes must be tested. It is suggested that the following durations applicable to the design of pipes draining both small and large areas be used: 3, 4, 5, 6, 7, 8, 10, 12, 14, 16, 18, 20, 25, ... up to durations for which the peak-flow values decrease. The rainfall intensity of part 2 can be obtained from, for example, the Intensity-Duration-Frequency relationships published by Dahlström (1979).

Rough estimates of the changes in return period due to changes in flow values can be made with the formula

$$F_2 = (Q_2/Q_1)^{1/z} \cdot F_1 \qquad \ldots(8.1)$$

where F_1 and F_2 are the return periods corresponding to the peak-flow values Q_1 and Q_2. For the examples given in this report z varied between 0.28 and 0.41, with a mean value of 0.34. Eq. (8.1) is not applicable in surcharged pipes or pipes influenced by backwater.

8.2 Recommendations for Further Research

The largest obstacle to a more common use of historical rainfall data and estimated local design storms is the lack of rainfall data or treated data. Therefore, it is of utmost importance to:

- Treat the rainfall data already available, including transfer of the data to a computer tape or disc, statistical analysis of the data, identification of major rainfall events, and estimation of local coefficients for design storms.

- Start new rainfall intensity measurements. At least one instrument should be running in each municipality. The best would be if the measurements could be coordinated by the Swedish Meteorological and Hydrological Institute (SMHI) and included in their planned network of automatic stations. However, while waiting for the automatic stations to go into operation, each user is recommended to start his own measurements. Each year is of great value because a long time series is needed before the data can be fully utilized, and there is still no final decision made to include rain-intensity measurements, with enough resolution in volume and time, at the automatic weather stations.

It is always difficult to generalize the results obtained for a few examples. They are valid only for runoff basins similar to the basins used in the examples. For other catchments there may be an influence on the peak flows also from antecedent precipitation and from contributions from the permeable areas. Further research is recommended on the following topics:

- Selection of rainfall data for areas of other sizes and with other structures of the sewer systems.
- The characteristics of the unit-hydrograph method, especially which rainfall intensity should be used for the estimation of the unit hydrographs in relation to the intensity of the historical rainfalls.
- The influence from antecedent precipitation on the simulated peak flows.
- Development of a "Swedish type of design storm." Since the design storms seem to give an acceptable design of the pipe systems, this may be worthwhile.

The results are valid only for areas of sizes up to a couple of hundred hectares. For larger areas the areal distribution of the individual rainfalls is of importance. For simpler runoff methods this can be taken into account by using so-called "fixed area, area reduction factors" (see Arnell _et_ _al_. 1980), and for the rational method different Intensity-Duration-Frequency curves can be estimated for different sizes of basins. The problem is larger when the areal distribution of time-varying rainfalls is needed for use in detailed mathematical runoff models, in which cases the same areal reduction factor probably cannot be applied to the individual intensity values as to the total rainfall. When a runoff model is used, the interest is usually focused not only on the outlet pipe in the sewer system but also on pipes upstream, draining smaller areas. Thus, the areal distribution for both small and large areas must be taken into account at the same time. When it is possible to do rainfall measurements, rainfall data from more than one instrument can be utilized in the runoff simulations, but when designing a new system, no such rainfall data are available. It must be stressed that the important thing is not in all cases to simulate individual runoff events correctly but to estimate the correct *statistical* design peak-flow values. Research is thus needed concerning:

- o The areal distribution of individual rainfall events and how to take that distribution into consideration when using distributed runoff models.

Dahlström (1979) has shown the variation in rainfall intensities across the whole of Sweden. However, one can suspect that the variation is significant also for smaller regions of up to a few hundred square kilometers. Especially in areas along the coast, there is a large variation in rainfall amounts. Investigations are therefore needed of:

- o The variation in rainfall intensities for smaller regions, especially along the coast, and the dependence of the topography, the distance to the coast, the "roughness" of the landscape, and the effect of urbanization.

The present report deals with the design of sewer pipes only. In many applications there is a need for the design of retention basins, overflows, pumping stations, etc. The operation of sewer systems in combination with treatment plants requires information about the rainfall intensities. Therefore, the research should continue with:

- o Studies of the rainfall data for the design of retention basins, overflows, pumping stations, and other parts of a storm-sewer system, including the operation of the system in combination with the treatment plant.

To make the results of different rainfall studies usable and well-known to the users, it is necessary to:

- o Write manuals and computer programs that make it easy to design sewer systems for various kinds of rainfall data.

- Inform about the selection of rainfall data for different applications and for different design methods.

The knowledge concerning many of the problems stated above can be increased if the work is focused on a more general model of rainfall climate, including the physics of the precipitation weather systems. This would be a large project, including not only conventional precipitation measurements but also radar- and satellite measurements and persons with different backgrounds such as meteorologists, mathematicians, statisticians, and engineers.

APPENDIX I

ESTIMATION OF A SIFALDA-TYPE DESIGN STORM

Method of Estimation

The Sifalda-type design storm was estimated by means of regression analyses of the durations of the maximum average-intensity parts of the rainfalls versus the other parameters. The logarithms of the durations were used when that gave a better correlation than the use of the values of the durations themselves. Part ③ of the storm was used for the regression analyses, because it gave a better correlation than the use of part ① . As input data in the regression analyses, data from Table 3.4 were used. The results of the regression analyses are given below, where the steps in the estimation of the storm are described. The different parameters are defined in Fig. I.1.

1. The total duration is estimated by the equation

$$T_{tot} = u + v \ln T \qquad \text{...(I.1)}$$

where T_{tot} = total duration of the rainfall (min)

T = duration of part ② , the maximum average-intensity part (min)

u, v = constants obtained from Table I.1

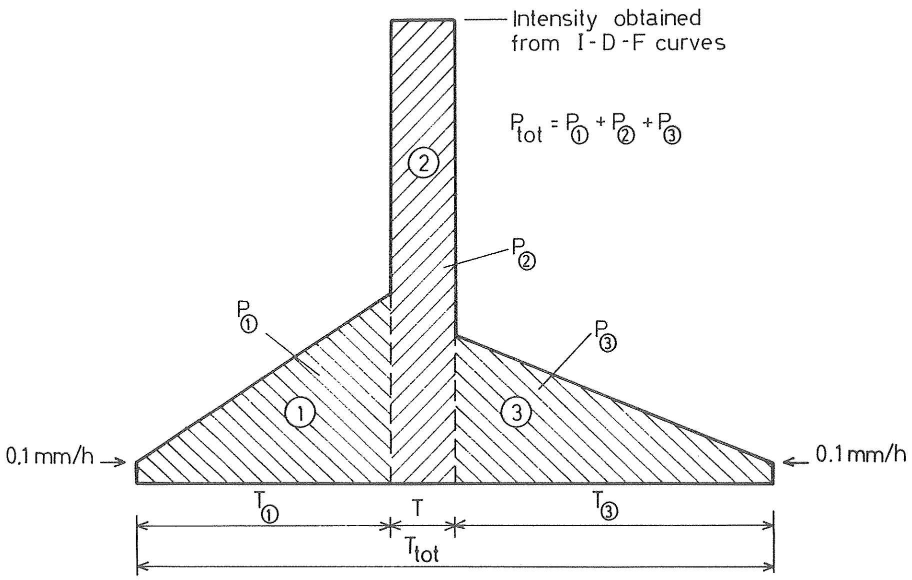


Fig. I.1 Definition of parameters of the Sifalda design storm.

2. The duration of part ③ is estimated by the equation

$$T_{③} = (e+f \cdot T)\ T_{tot} \cdot \frac{1}{100} \qquad \text{...(I.2)}$$

where $T_{③}$ = duration of part ③ (min)

e, f = constants obtained from Table I.2.

The parenthesis of Eq. (I.2) gives the duration of part ③ as a percentage of the total duration.

3. The duration of part ① is calculated by the equation

$$T_{①} = T_{tot} - T - T_{③} \qquad \text{...(I.3)}$$

where $T_{①}$ = duration of part ① (min)

4. The total volume of the rainfall is calculated by the equation

$$P_{tot} = g + h \cdot \ln T \qquad \text{...(I.4)}$$

where P_{tot} = total volume of the rainfall (mm)

g, h = constants obtained from Table I.3.

5. The volume of part ③ is estimated by the equation

$$P_{③} = (k + l \cdot T)\ P_{tot} \cdot \frac{1}{100} \qquad \text{...(I.5)}$$

where $P_{③}$ = volume of part ③ (mm)

k, l = constants obtained from Table I.4.

The parenthesis of Eq. (I,5) gives the volume of part ③ as a percentage of the total volume.

6. The volume of part ① is calculated by the equation

$$P_{①} = P_{tot} - P_{②} - P_{③} \qquad \text{...(I.6)}$$

where $P_{①}$ = volume of part ① (mm)

$P_{②}$ = volume of part ② obtained from the I-D-F curves (mm)

7. The distribution in time of the volumes is done when the durations and the volumes of the different parts are known, and with the added restriction that the intensities at the beginning and at the end of the rainfall are selected to be 0.1 mm/h. This intensity value is equal to the value used for the separation of the independent rainfalls (see Section 5.3).

Table I.1 *Values of the constants u and v in the equation* T_{tot} *(min)* $= u+v \cdot \ln T$ *and values of the correlation coefficient. Lundby, Göteborg, 1921-1939.*

F year	u	v	r_s
1/3	291.67	46.77	0.91
1/2	238.07	55.67	0.94
1	125.25	73.09	0.94
2	111.17	62.49	0.76
5	74.34	70.85	0.84

Table I.2 *Values of the constants e and f in the equation* T_{3} *(%)* $= e+f \cdot T$ *and values of the correlation coefficient. Lundby, Göteborg, 1921-1939.*

F year	e	f	r_s
1/3	56.73	-0.11	-0.98
1/2	59.72	-0.14	-0.98
1	76.40	-0.24	-0.97
2	75.31	-0.24	-0.94
5	67.70	-0.19	-0.92

Table I.3 *Values of the constants g and h in the equation P_{tot} (mm) = $g+h\cdot \ln T$ and values of the correlation coefficient. Lundby, Göteborg, 1921-1939.*

F year	g	h	r_s
1/3	11.07	2.84	0.99
1/2	9.82	3.57	0.99
1	7.37	4.90	0.98
2	8.92	5.04	0.96
5	8.99	5.58	0.94

Table I.4 *Values of the constants k and l in the equation P_{3} (%) = $k+l\cdot T$ and values of the correlation coefficient. Lundby, Göteborg, 1921-1939.*

F year	k	l	r_s
1/3	35.33	-0.12	-0.94
1/2	34.12	-0.11	-0.97
1	30.09	-0.09	-0.93
2	25.65	-0.07	-0.85
5	18.26	-0.03	-0.41

APPENDIX II

RESULT OF AN EVALUATION OF THE CUMULATIVE PRECIPITATION AS A FUNCTION OF THE CUMULATIVE STORM TIME

Method of Evalution

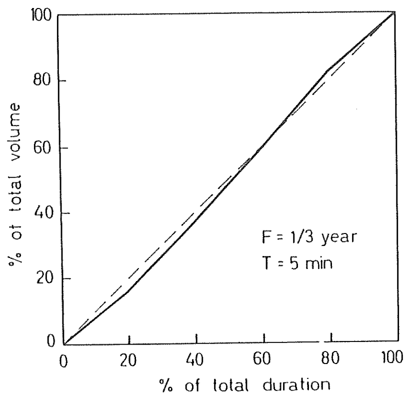
For each duration from 5 minutes to 240 minutes of the maximum average-intensity period, the historical rainfalls were identified for which the maximum average-intensity value was larger than the value corresponding to a specified return period (values obtained from Table 2.2). This gave one group of rainfalls for each return period and for each duration.

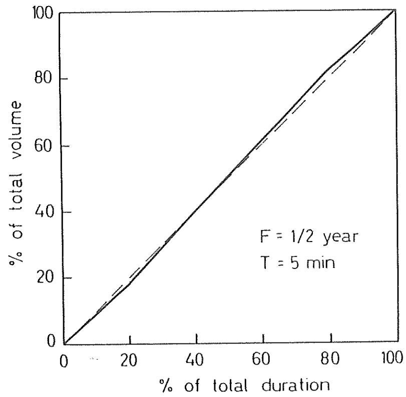
The number of rainfalls for each duration was for a return period of 1/3 year 54, 1/2 year 36, 1 year 18, 2 years 9, and 5 years 6 rainfalls. The evaluations were then made for the durations of 5, 10, 15, 30, 60, 120, and 240 minutes.

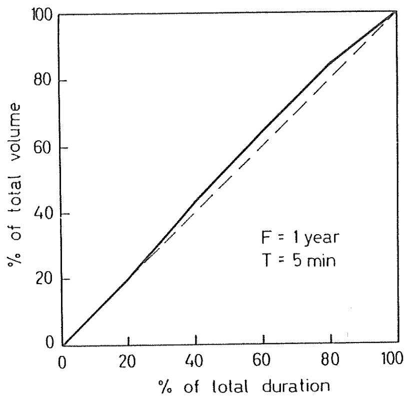
For all historical rainfalls of each return period and of each duration, the temporal rainfall pattern was evaluated. The evaluation was done within each maximum average-intensity period, and the result was expressed in cumulative percent of precipitation as a function of cumulative percent of storm time, and average values were then calculated for each return period and duration, and the results are shown below. The dashed line with a slope of 1:1 is shown for comparison.

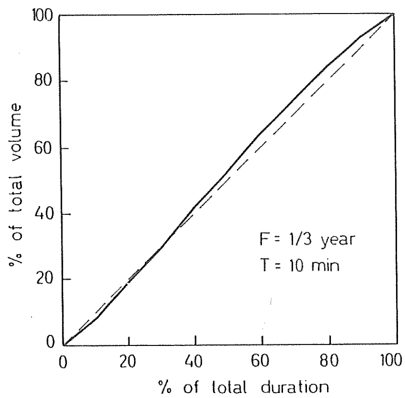
The curves were grouped into four categories based on a judgment of the shapes of the curves. One average curve for each category was calculated and is shown in Fig. 3.9.

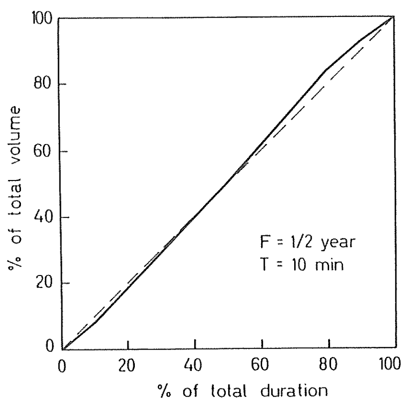
Resulting cumulative precipitation as a function of cumulative storm time. $1/3 \le F \le 1$ year, $5 \le T \le 20$ minutes

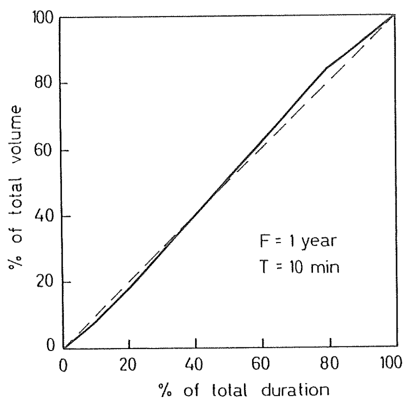


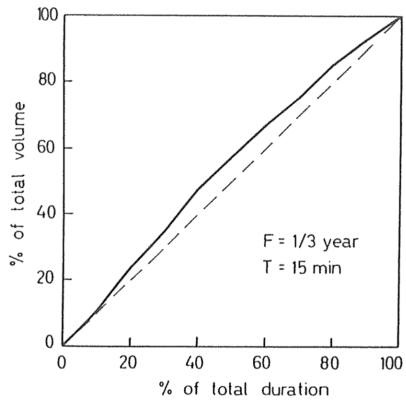


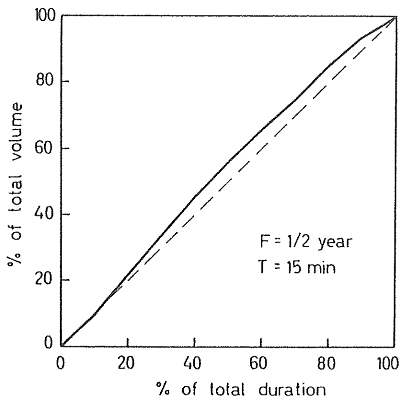


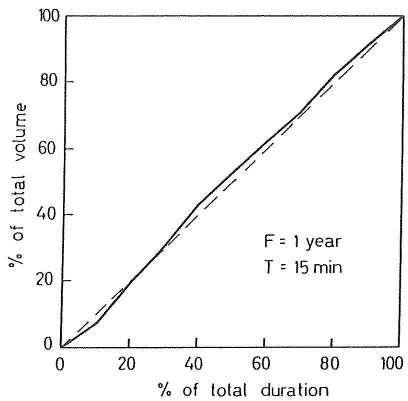




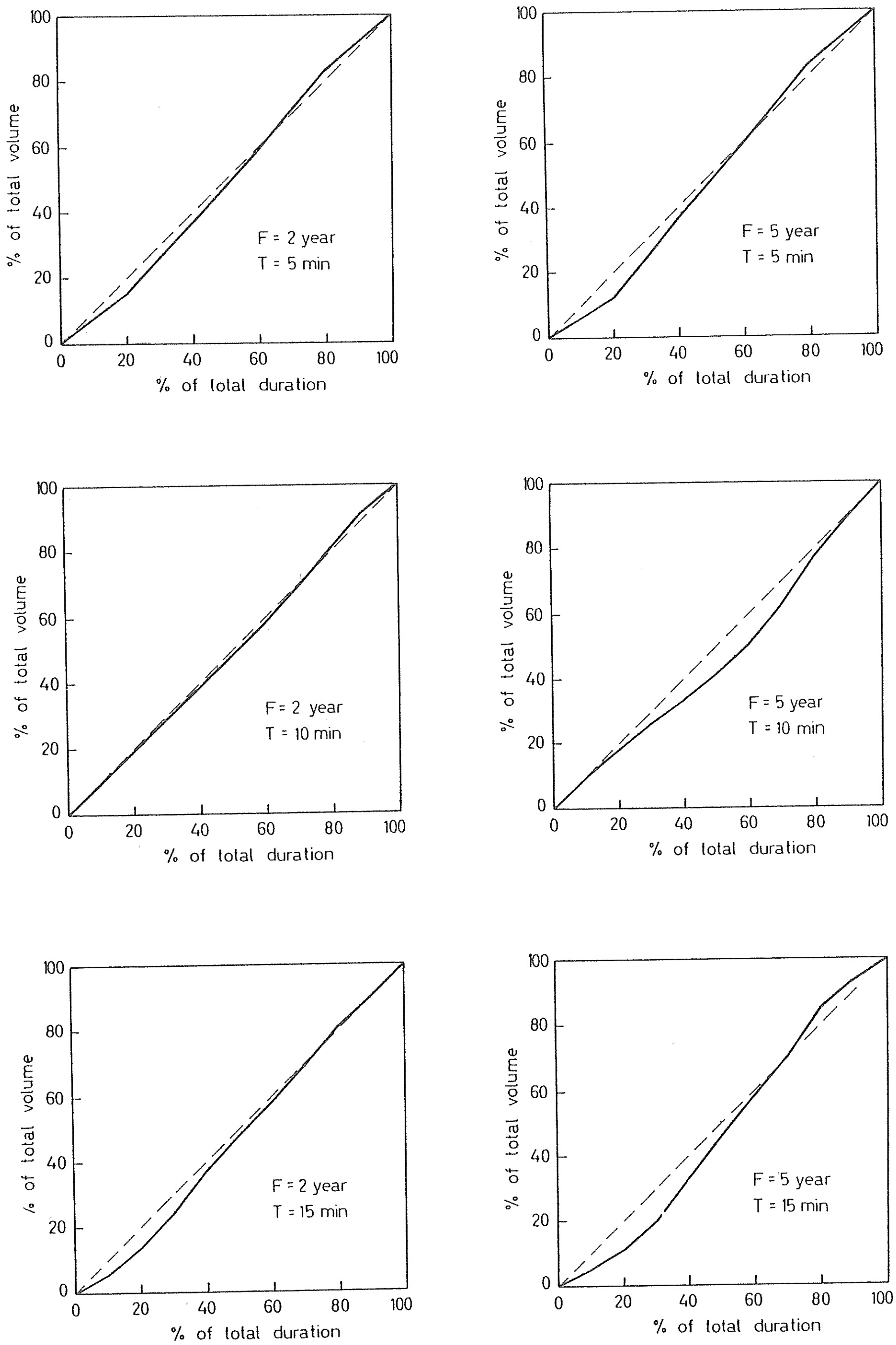



100
80
60
40
20
0
0
20
40
60
80
100
% of total volume
% of total duration
F = 1/3 year
T = 15 min

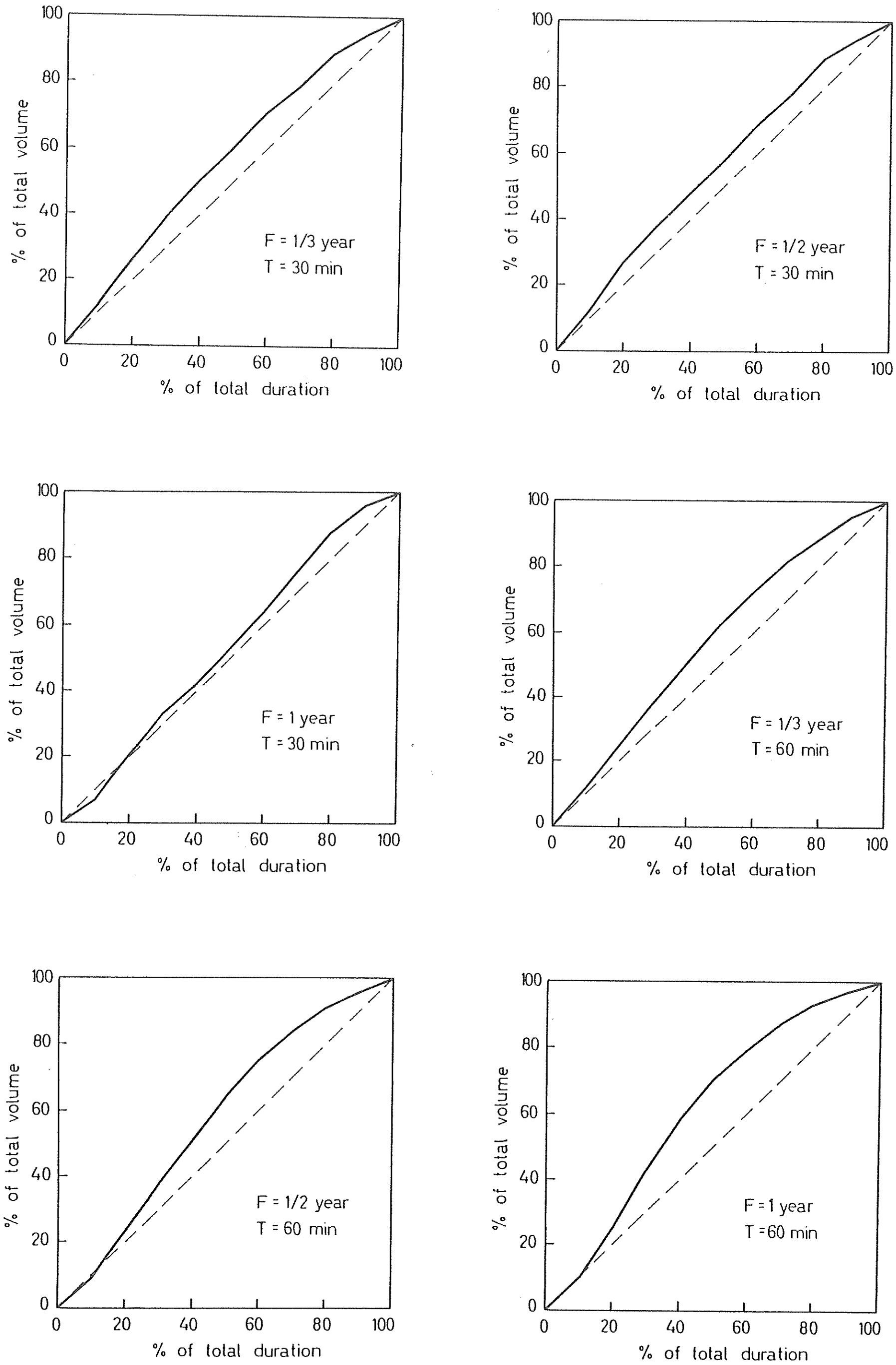

100
80
60
40
20
0
0
20
40
60
80
100
% of total volume
% of total duration
F = 1/2 year
T = 15 min

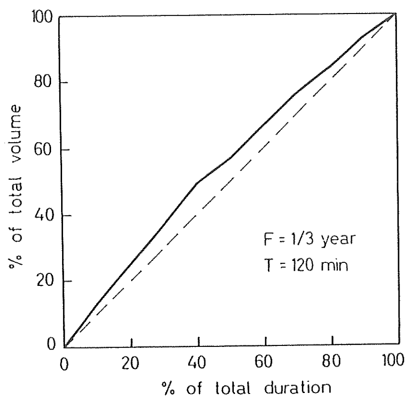

100
80
60
40
20
0
0
20
40
60
80
100
% of total volume
% of total duration
F = 1 year
T = 15 min

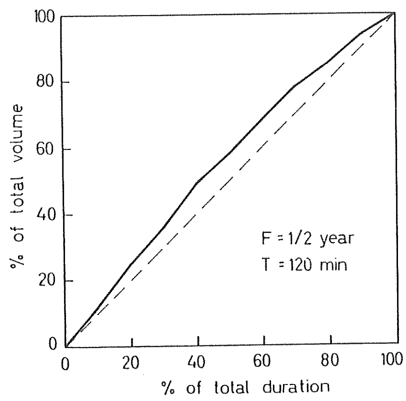
Resulting cumulative precipitation as a function of cumulative storm time. 2 $\leq$ F $\leq$ 5 years, 5 $\leq$ T $\leq$ 20 minutes

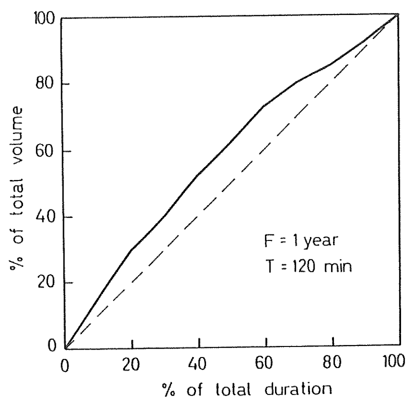


Resulting cumulative precipitation as a function of cumulative storm time. $1/3 \leq F \leq 1$ year, $25 \leq T \leq 170$ minutes

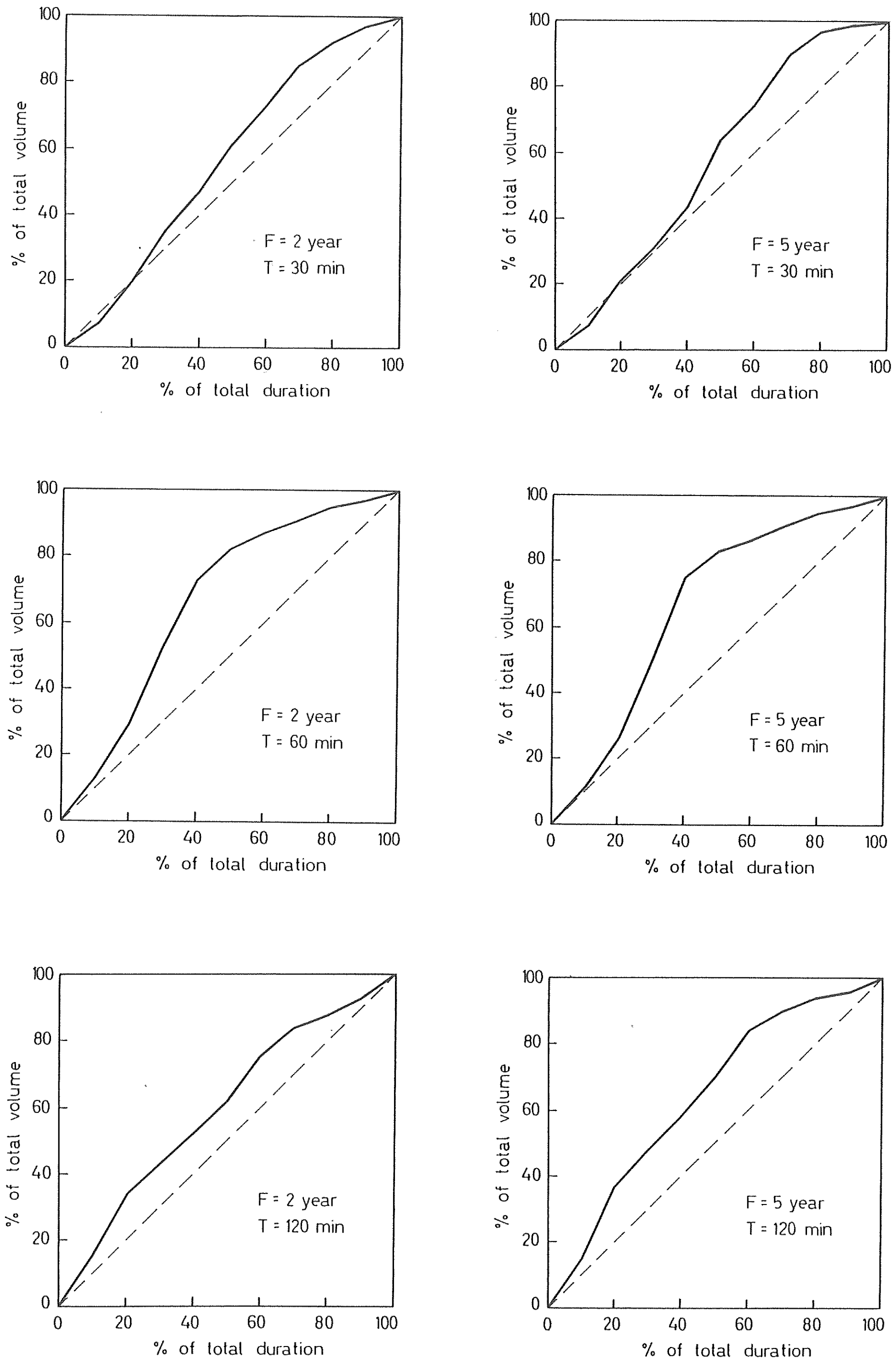



100
80
60
40
20
0
% of total volume
F = 1/3 year
T = 120 min
0
20
40
60
80
100
% of total duration

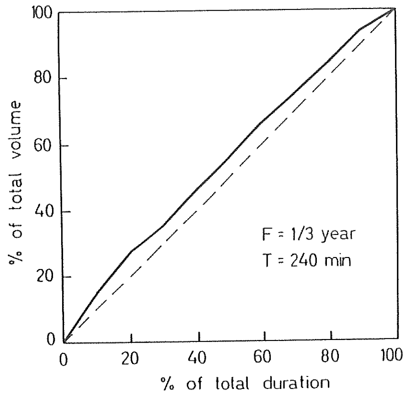

100
80
60
40
20
0
% of total volume
F = 1/2 year
T = 120 min
0
20
40
60
80
100
% of total duration

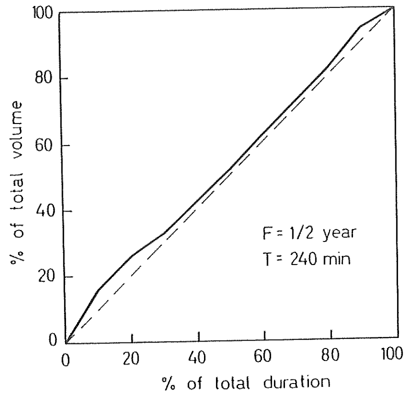

100
80
60
40
20
0
% of total volume
F = 1 year
T = 120 min
0
20
40
60
80
100
% of total duration

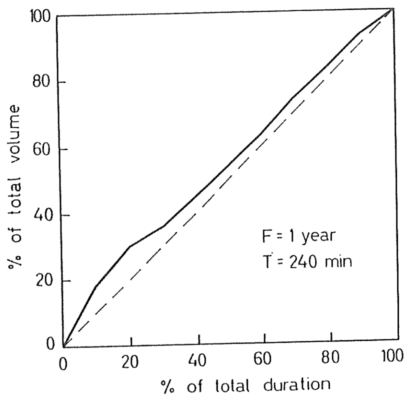
Resulting cumulative precipitation as a function of cumulative storm time. $2 \leq F \leq 5$ years, $25 \leq T \leq 170$ minutes

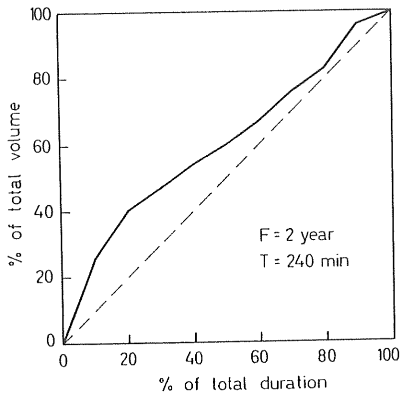


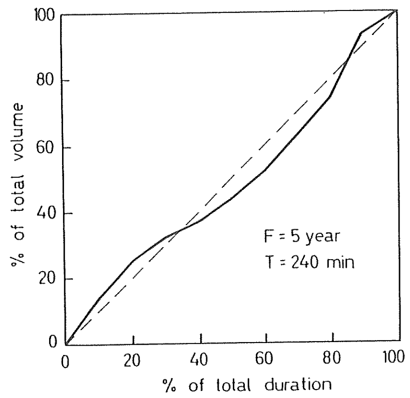
Resulting cumulative precipitation as a function of cumulative storm time. 1/3 ≤ F ≤ 5 years, T = 240 minutes











APPENDIX III

STATISTICAL DISTRIBUTIONS FOR CALCULATED PEAK FLOWS

Plotting of Calculated Peak Flows for Historical Storms and Different Design Storms

After calculation of the peak flows with the CTH-Model for the historical storms, the 36 largest peak flows for each pipe were ranked in descending order and plotted on exponential distribution papers with a logarithmic scale. The following formula was used for the estimation of the plotting positions

$$y_i = \sum_{j=1}^{i} \frac{1}{N+1-j}; \qquad i = 1, 2, \ldots, N \qquad \ldots(\text{III.1})$$

where y_i = plotting positions for the calculated peak-flow values in increasing order.

N = number of treated peak-flow values which are chosen as equal to the number of treated time periods, in this case 36 1/2-year periods.

y_i is related to the return period through the relationship

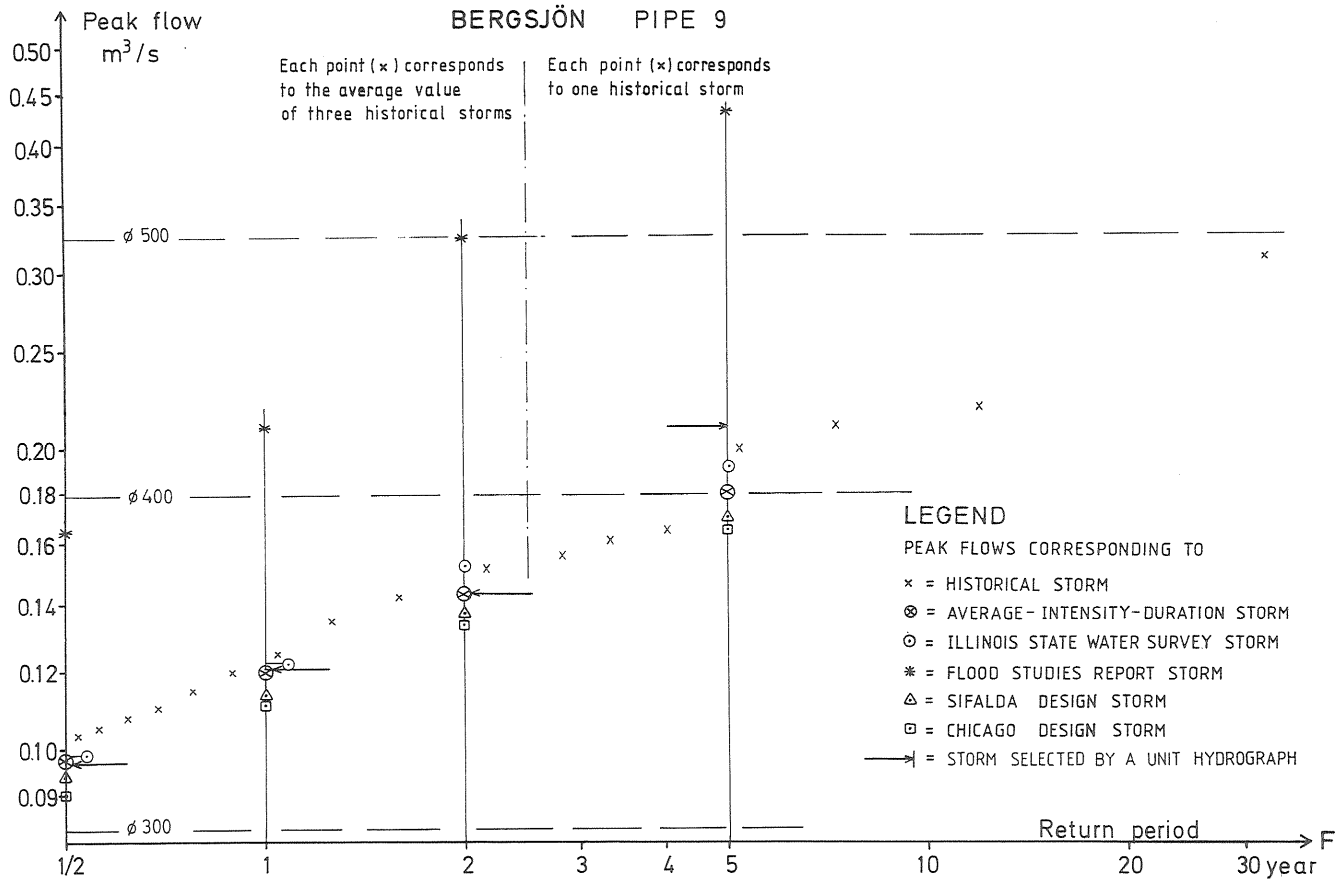
$$y_i = \ln F \qquad \ldots(\text{III.2})$$

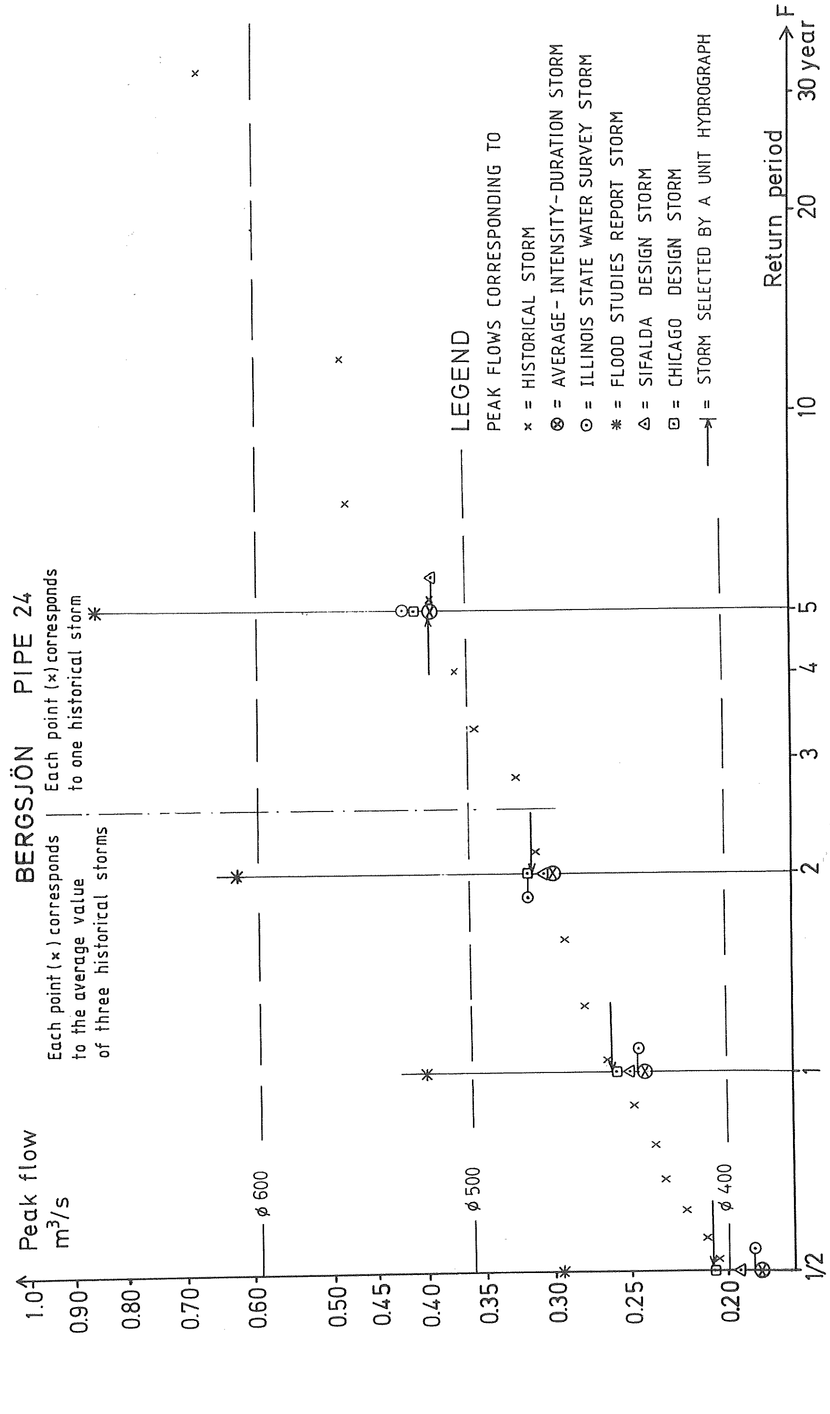
where F = return period in number of treated time periods, in this case 1/2-year periods (in the figures, F is given in years).

In order to make the plottings more clear, the 7 largest flow values were plotted as individual points and the remaining flow values were plotted as average values of 3 flow values and 3 y-values.

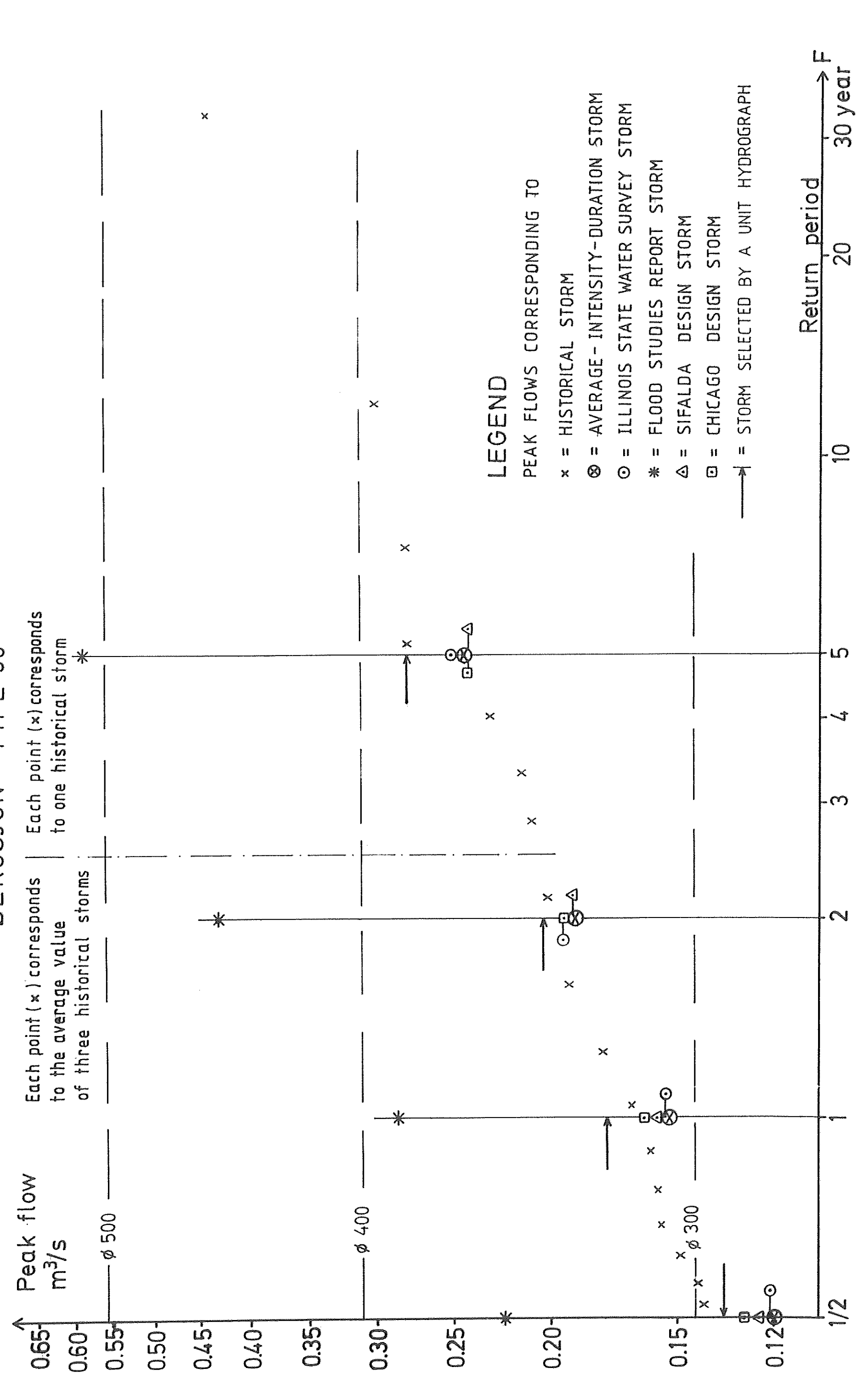
The peak flows for the different design storms were plotted in the diagrams for the return periods 1/2, 1, 2, and 5 years.

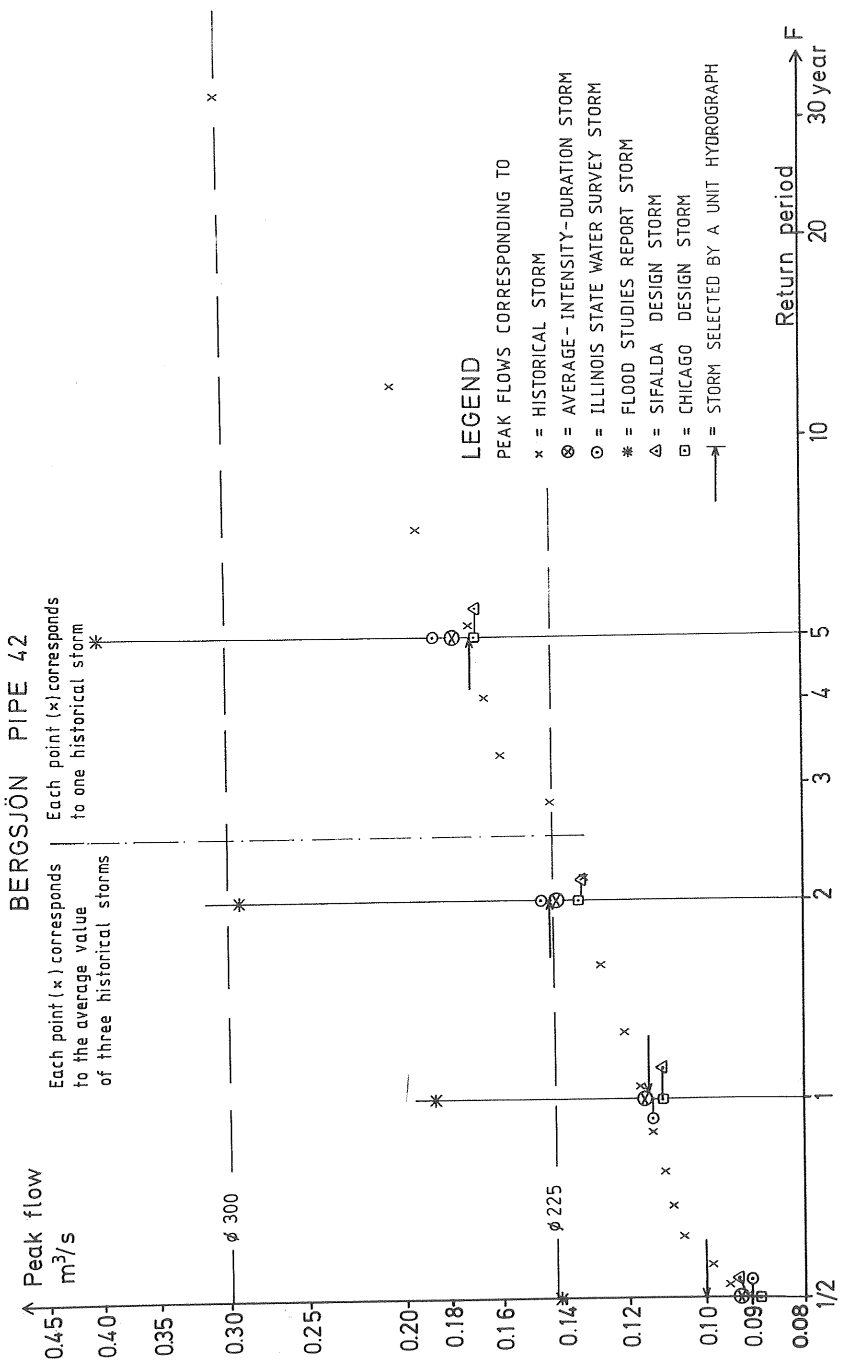
In the diagrams also the flow capacities for different standard diameters of the pipes are marked, and for the slopes of the individual pipes. As roughness coefficient a value of Manning's coefficient of 0.012 was used, corresponding to an effective absolute roughness of 1.0 mm (assuming a water temperature of $10^{\circ}C$).

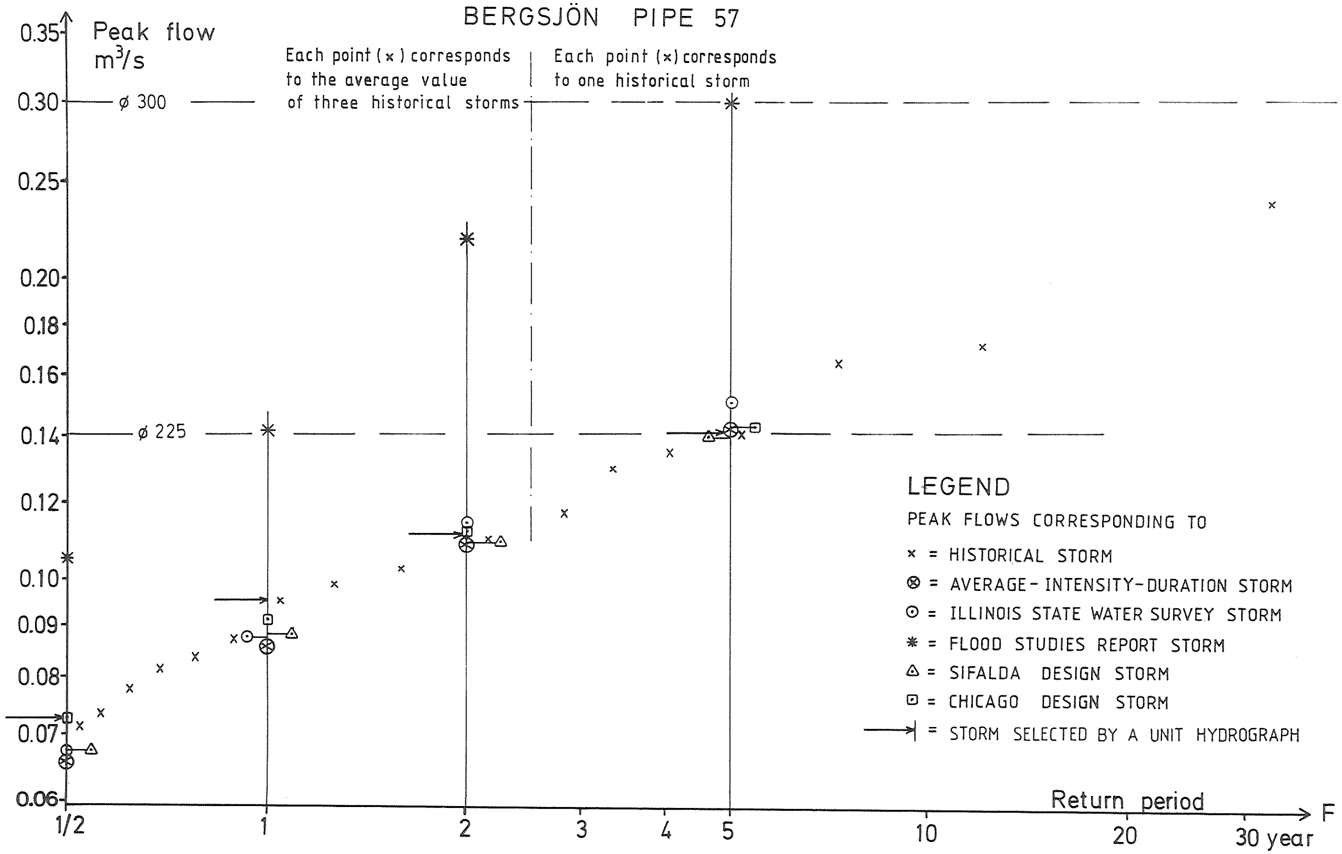

BERGSJÖN PIPE 9
Peak flow m³/s
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.18
0.16
0.14
0.12
0.10
0.09
ø 500
ø 400
ø 300
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH
Return period
F
1/2
1
2
3
4
5
10
20
30 year

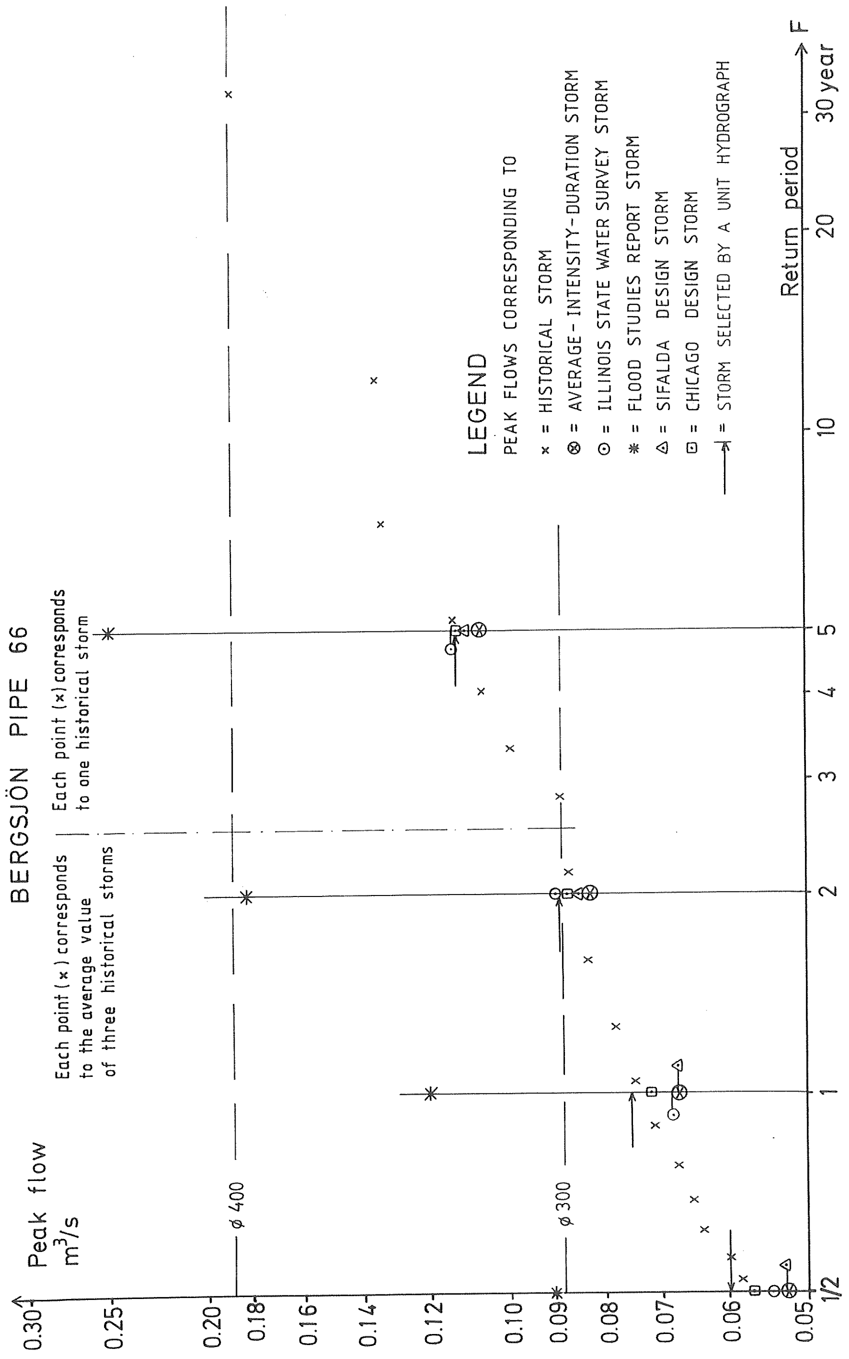


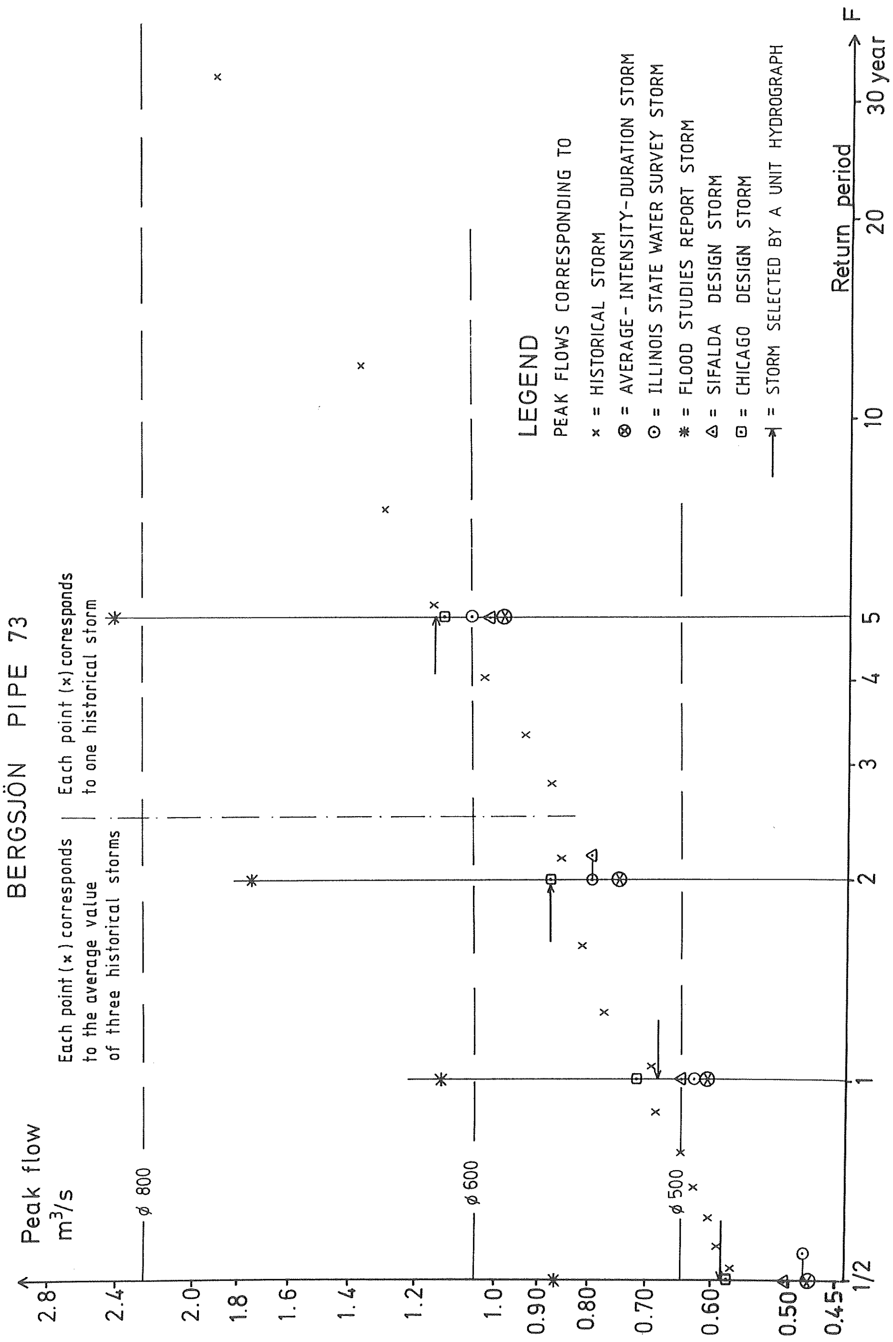
BERGSJÖN PIPE 24
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
Peak flow
m³/s
1.0
0.90
0.80
0.70
0.60
0.50
0.45
0.40
0.35
0.30
0.25
0.20
ø 600
ø 500
ø 400
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH


BERGSJÖN PIPE 30
Peak flow m³/s
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
ϕ 500
ϕ 400
ϕ 300
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.12
1/2
1
2
3
4
5
10
20
30 year
F
Return period
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE - INTENSITY - DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

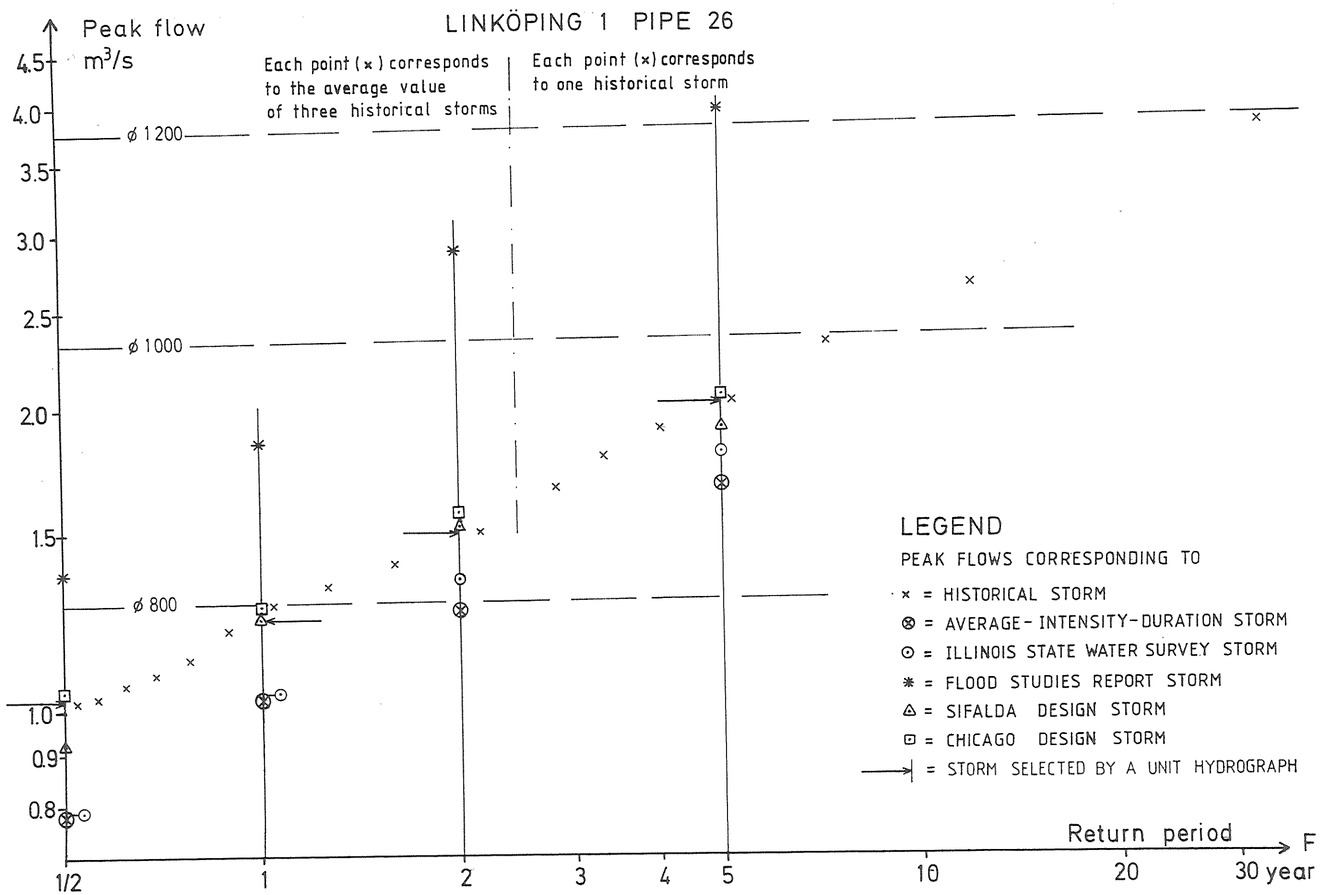

BERGSJÖN PIPE 42
Peak flow
m³/s
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
ø 300
ø 225
0.45
0.40
0.35
0.30
0.25
0.20
0.18
0.16
0.14
0.12
0.10
0.09
0.08
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✱ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

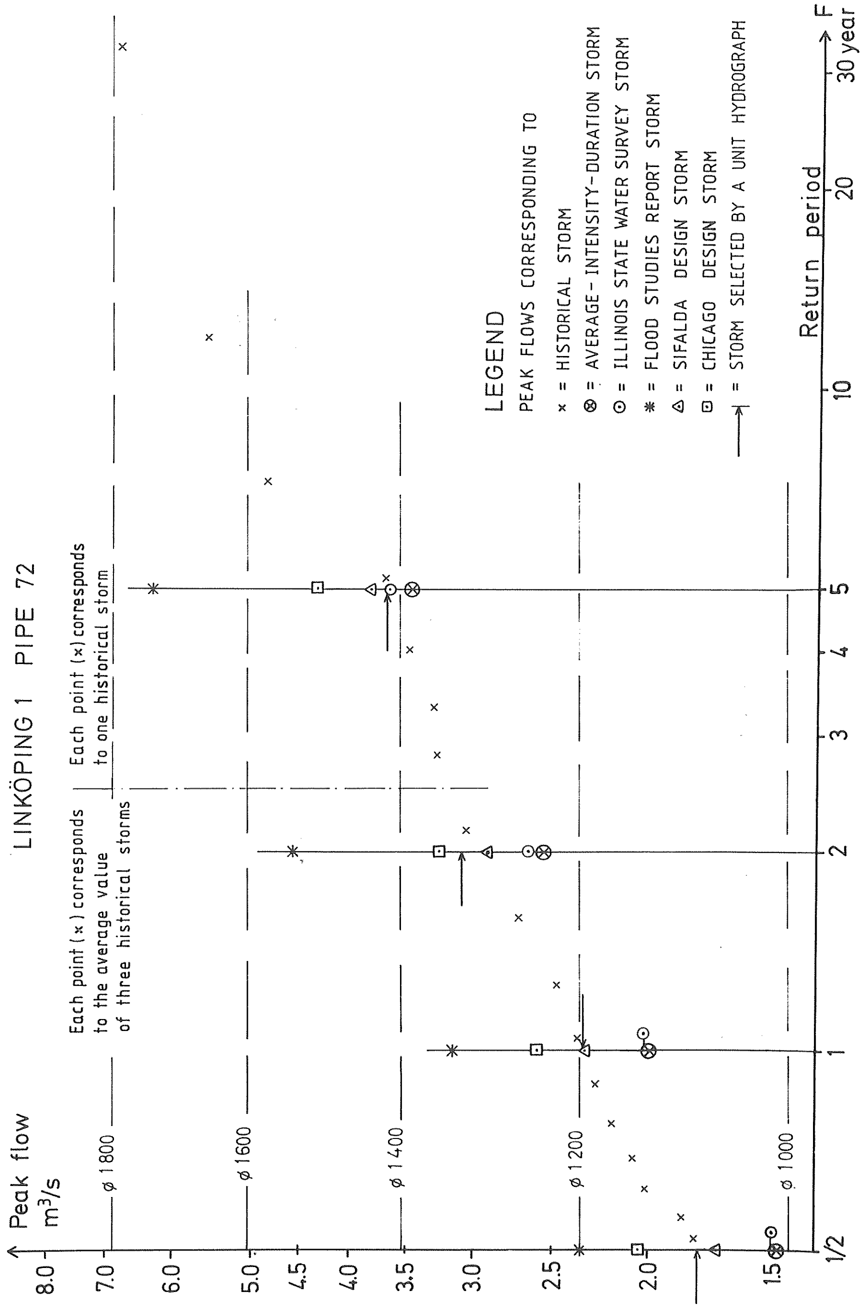

BERGSJÖN PIPE 57
Peak flow
m³/s
Each point (×) corresponds
to the average value
of three historical storms
Each point (×) corresponds
to one historical storm
ø 300
ø 225
0.35
0.30
0.25
0.20
0.18
0.16
0.14
0.12
0.10
0.09
0.08
0.07
0.06
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
→| = STORM SELECTED BY A UNIT HYDROGRAPH

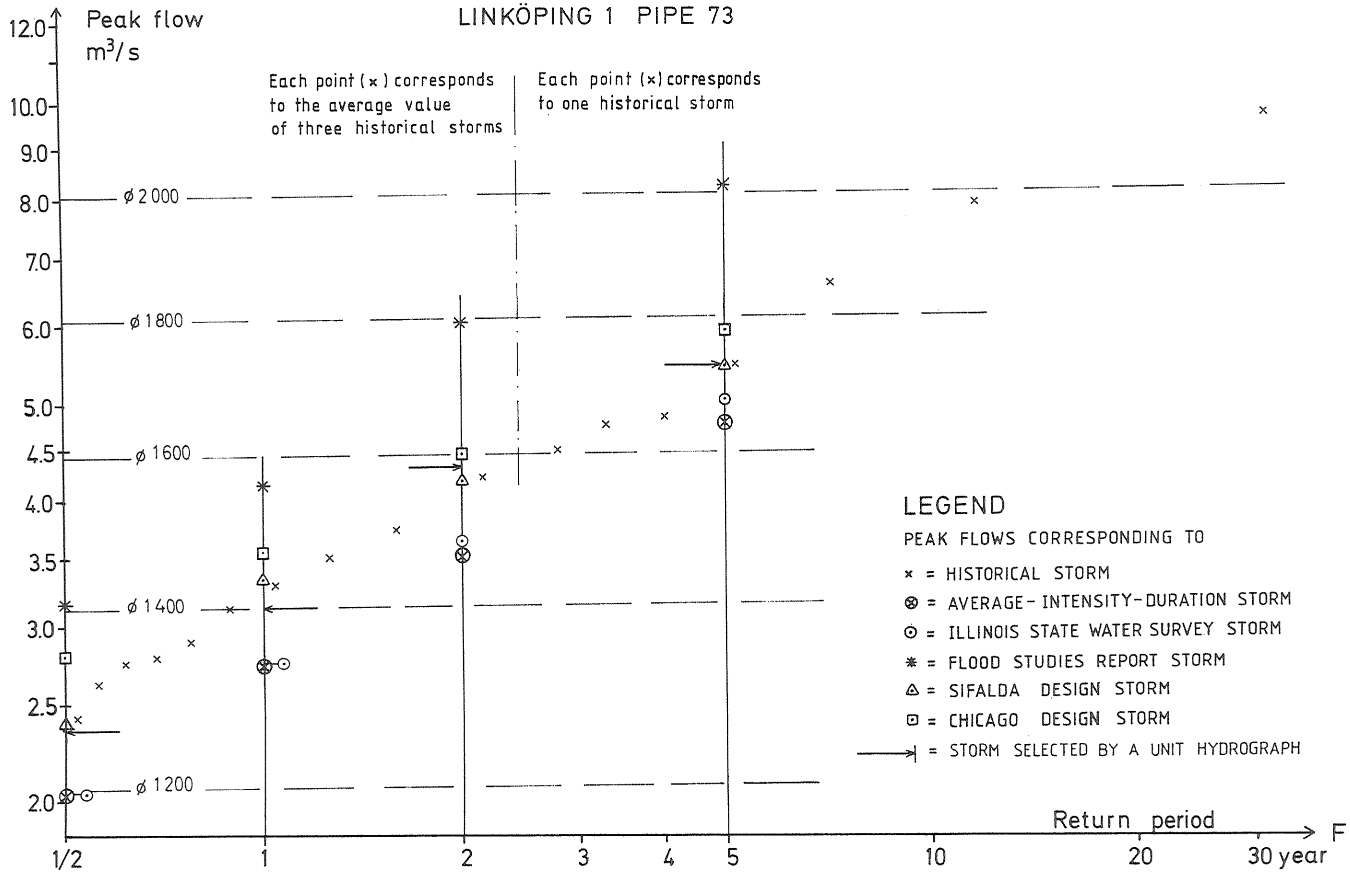

BERGSJÖN PIPE 66
Peak flow
m³/s
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
ø 400
ø 300
0.30
0.25
0.20
0.18
0.16
0.14
0.12
0.10
0.09
0.08
0.07
0.06
0.05
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE - INTENSITY - DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

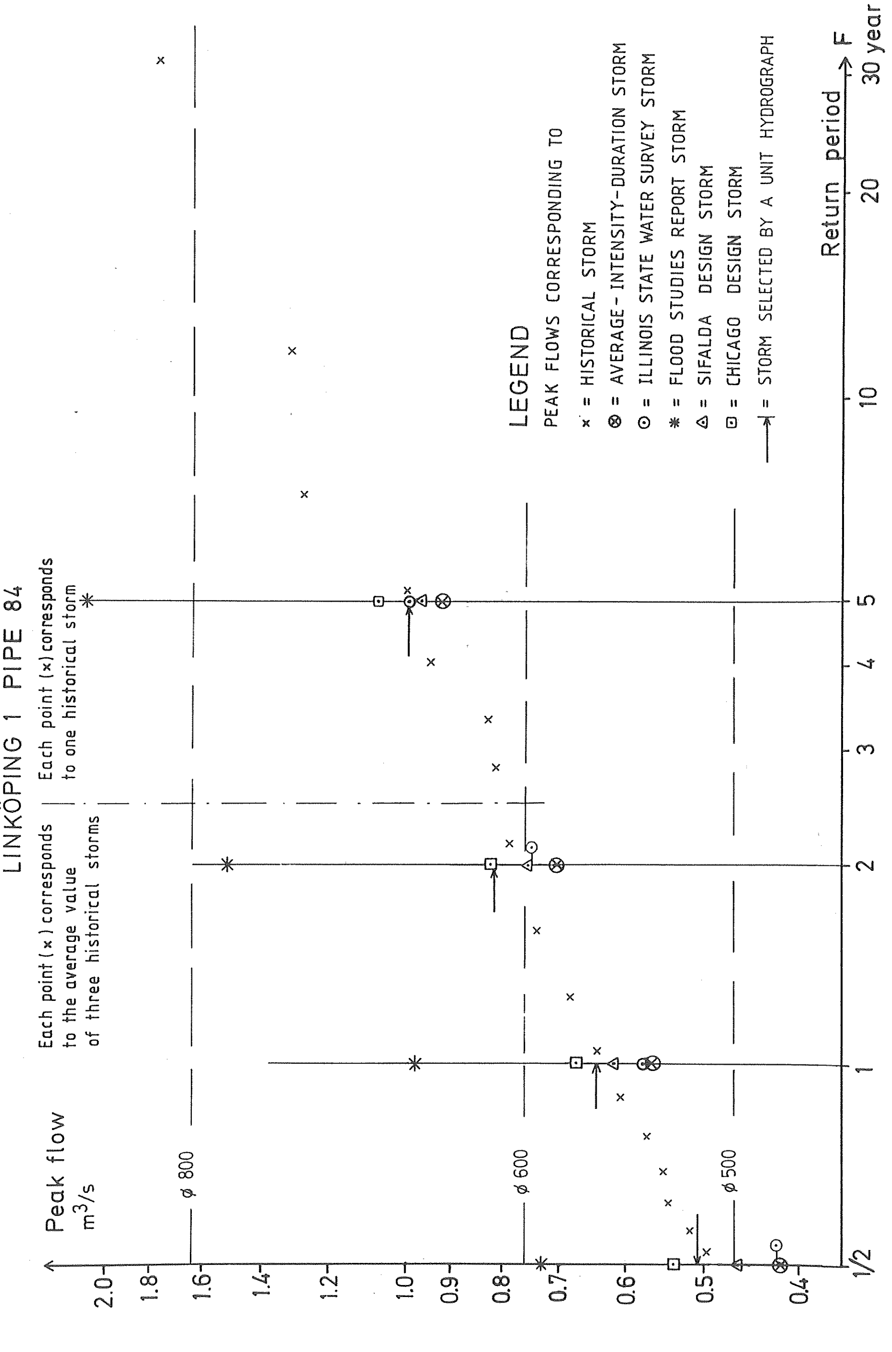


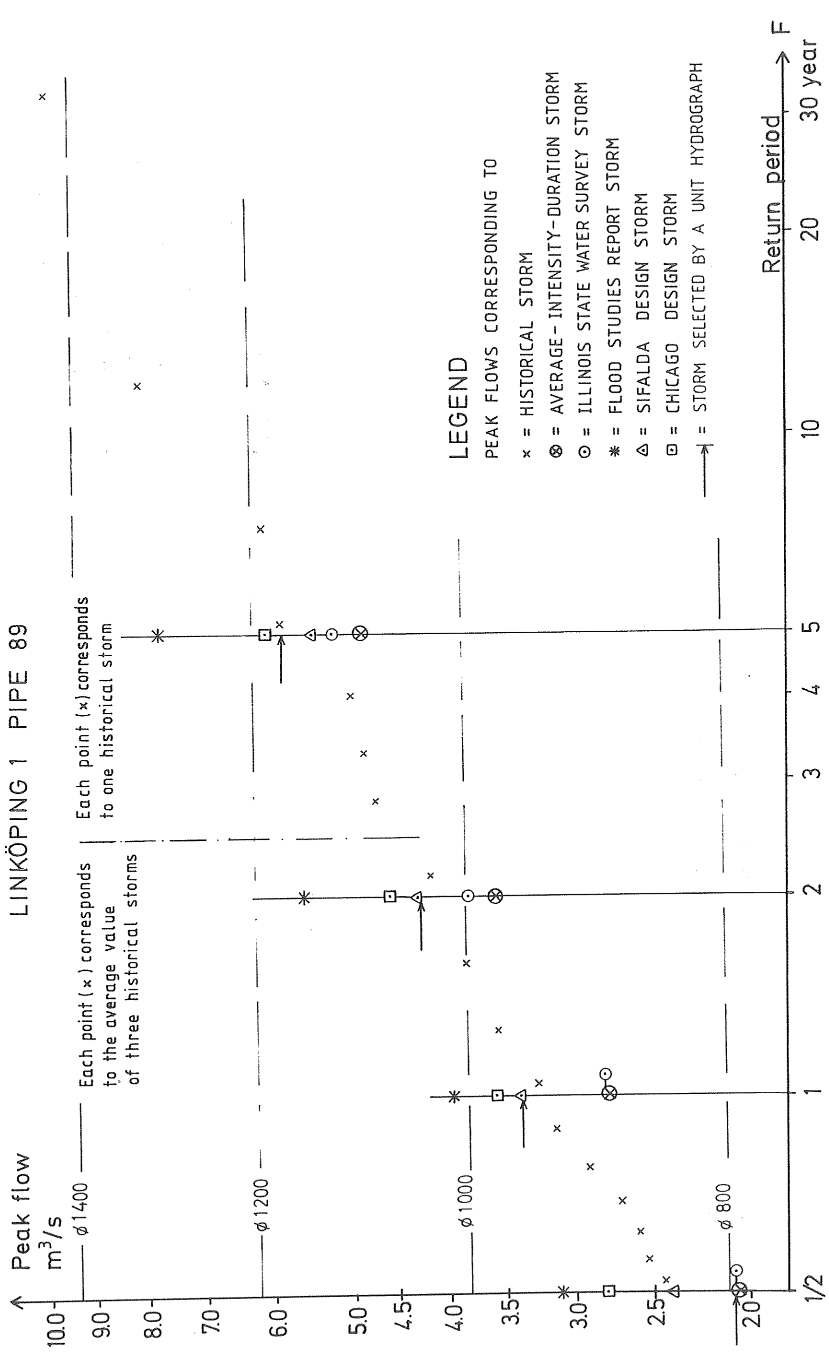
BERGSJÖN PIPE 73
Peak flow m³/s
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
ø 800
ø 600
ø 500
2.8
2.4
2.0
1.8
1.6
1.4
1.2
1.0
0.90
0.80
0.70
0.60
0.50
0.45
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
→ = STORM SELECTED BY A UNIT HYDROGRAPH


LINKÖPING 1 PIPE 26
Peak flow
m³/s
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.9
0.8
ø 1200
ø 1000
ø 800
Each point (×) corresponds
to the average value
of three historical storms
Each point (×) corresponds
to one historical storm
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH
Return period
F
1/2
1
2
3
4
5
10
20
30 year

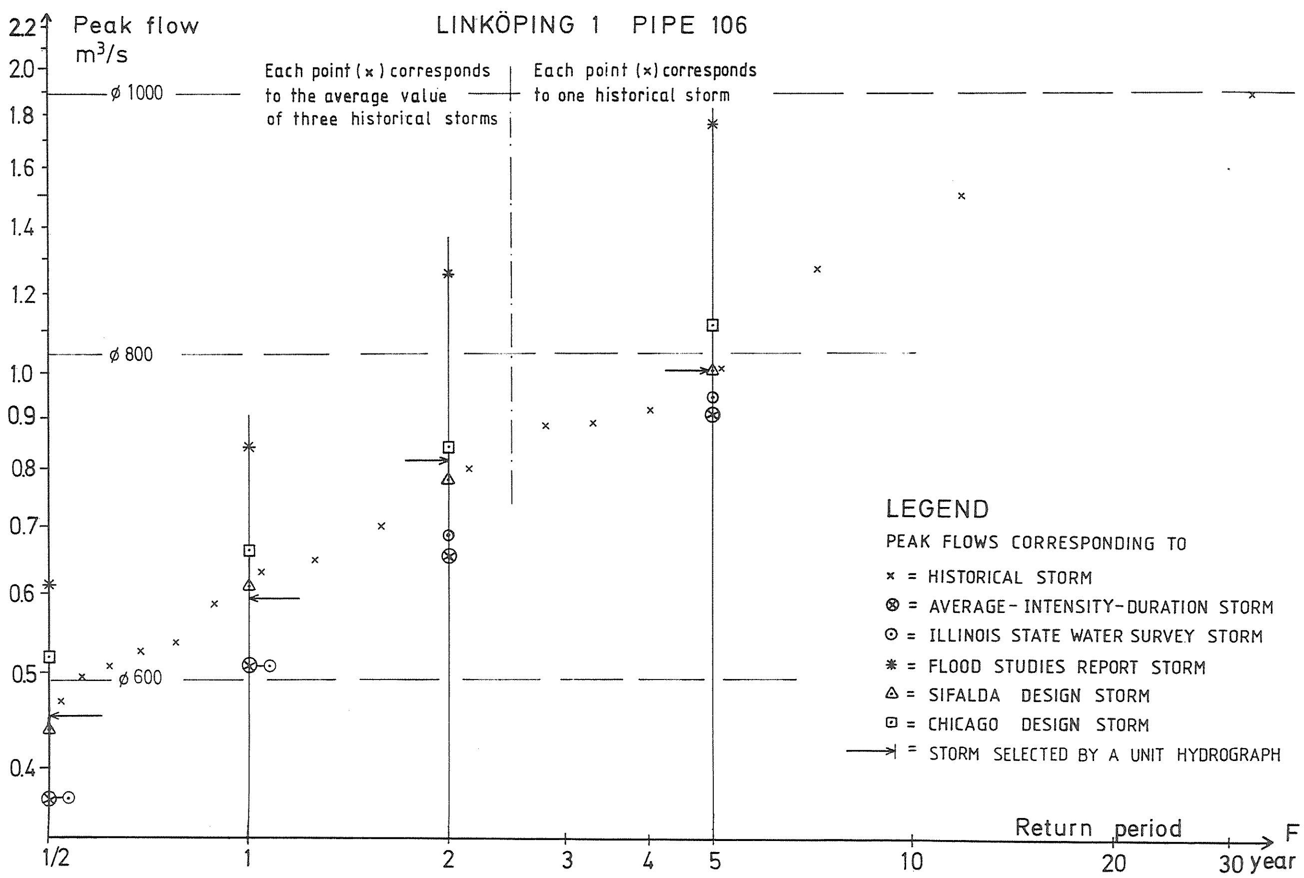

LINKÖPING 1 PIPE 72
Peak flow
m³/s
8.0
7.0
6.0
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
ø 1800
ø 1600
ø 1400
ø 1200
ø 1000
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
→ = STORM SELECTED BY A UNIT HYDROGRAPH

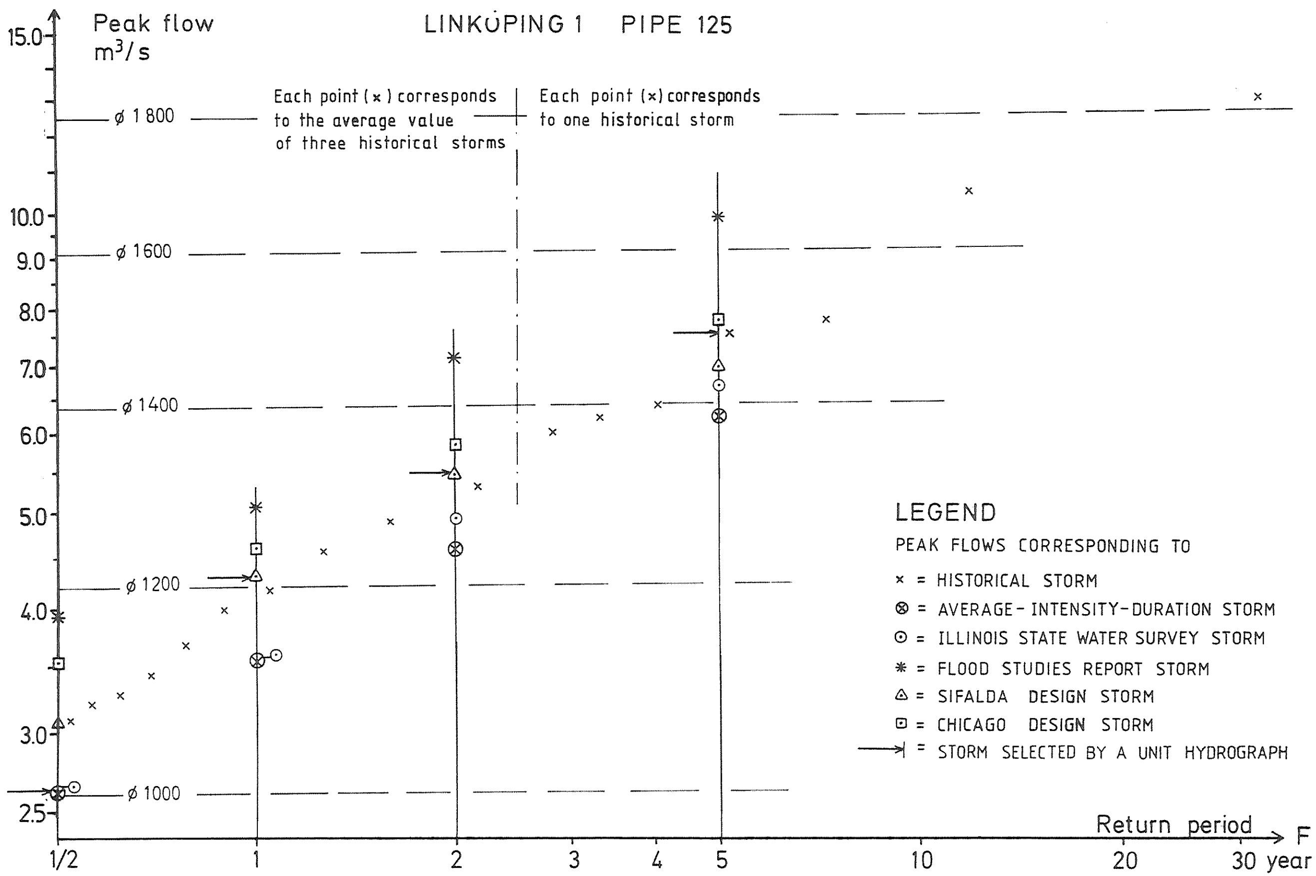

LINKÖPING 1 PIPE 73
Peak flow
m³/s
12.0
10.0
9.0
8.0
7.0
6.0
5.0
4.5
4.0
3.5
3.0
2.5
2.0
Each point (×) corresponds
to the average value
of three historical storms
Each point (×) corresponds
to one historical storm
ø 2000
ø 1800
ø 1600
ø 1400
ø 1200
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH
Return period
F
1/2
1
2
3
4
5
10
20
30 year


LINKÖPING 1 PIPE 84
Peak flow
m³/s
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
ø 800
ø 600
ø 500
2.0
1.8
1.6
1.4
1.2
1.0
0.9
0.8
0.7
0.6
0.5
0.4
1/2
1
2
3
4
5
10
20
30 year
Return period F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE - INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

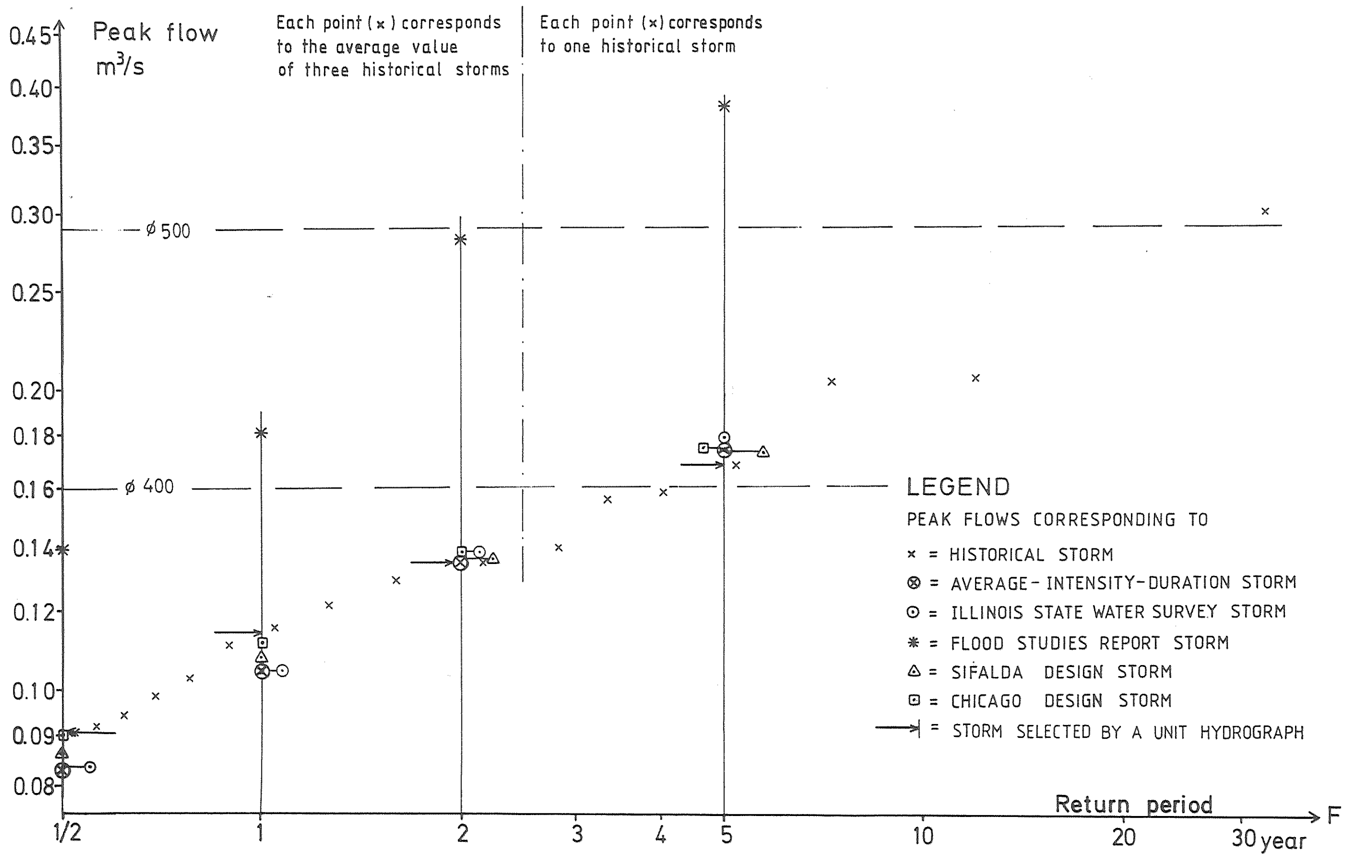


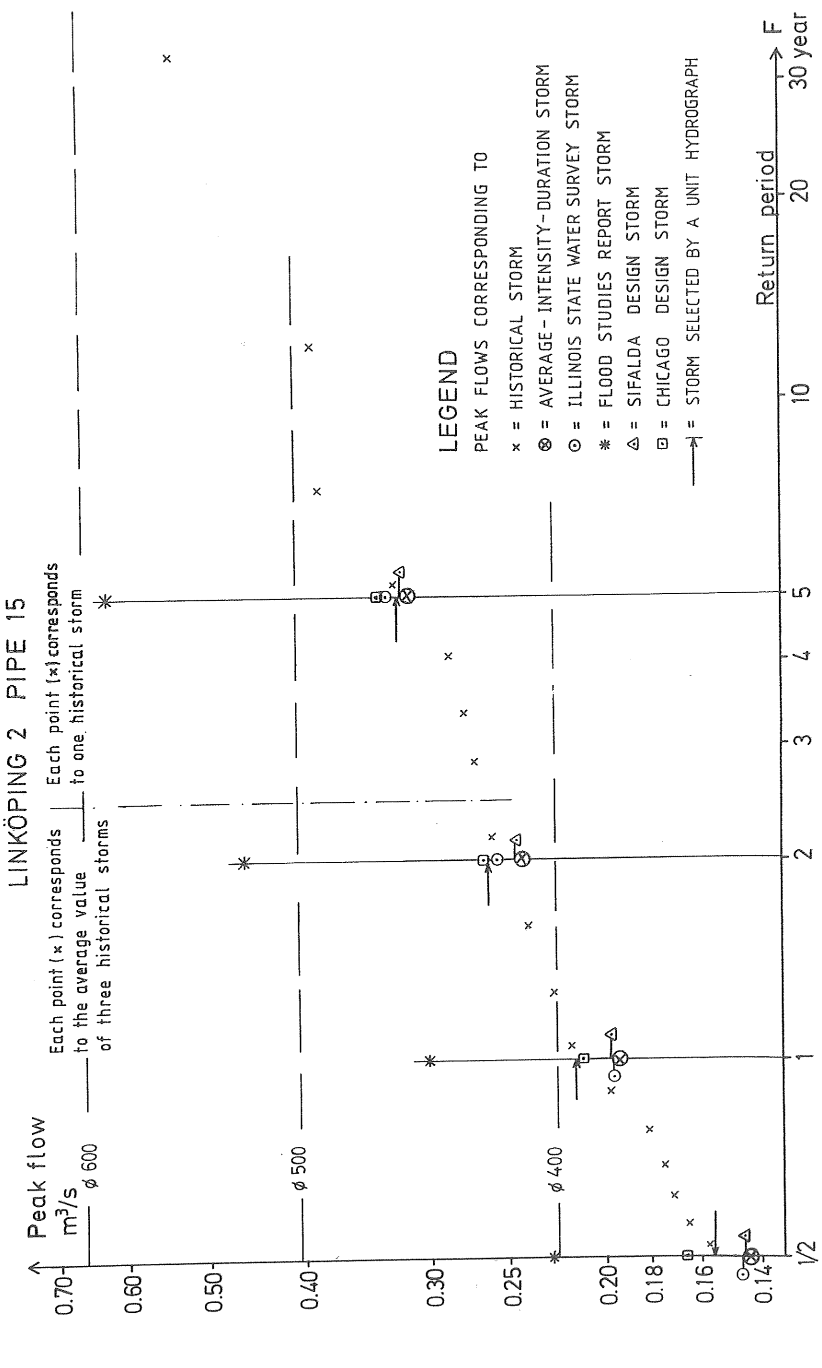
LINKÖPING 1 PIPE 89
Peak flow
m³/s
10.0
9.0
8.0
7.0
6.0
5.0
4.5
4.0
3.5
3.0
2.5
2.0
ø 1400
ø 1200
ø 1000
ø 800
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
PEAK FLOWS CORRESPONDING TO
x = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH
1/2
1
2
3
4
5
10
20
30 year
Return period
F

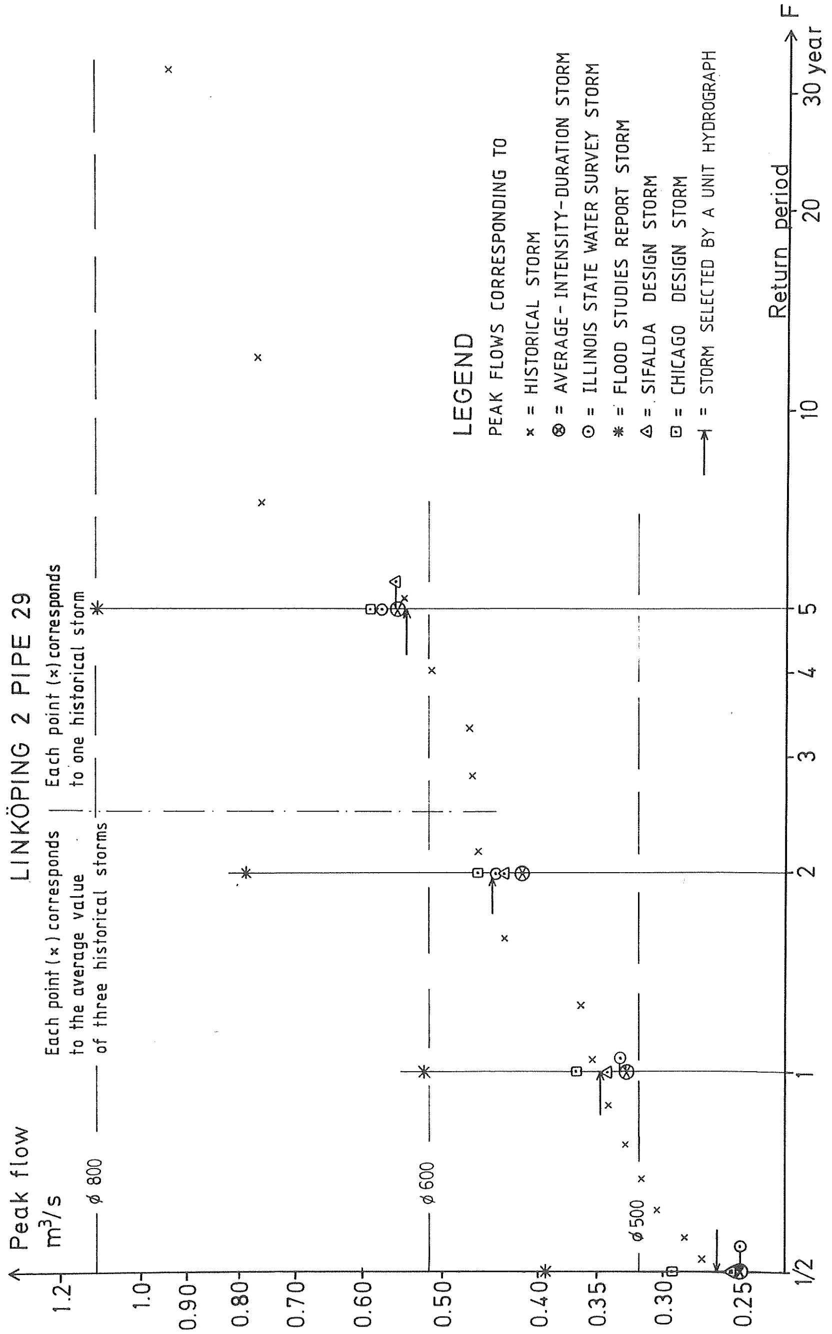

LINKÖPING 1 PIPE 106
Peak flow m³/s
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.9
0.8
0.7
0.6
0.5
0.4
ø 1000
ø 800
ø 600
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH
Return period F
1/2
1
2
3
4
5
10
20
30 year


LINKÖPING 1 PIPE 125
Peak flow
m³/s
15.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.5
ø 1800
ø 1600
ø 1400
ø 1200
ø 1000
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
→| = STORM SELECTED BY A UNIT HYDROGRAPH
Return period
F
1/2
1
2
3
4
5
10
20
30 year

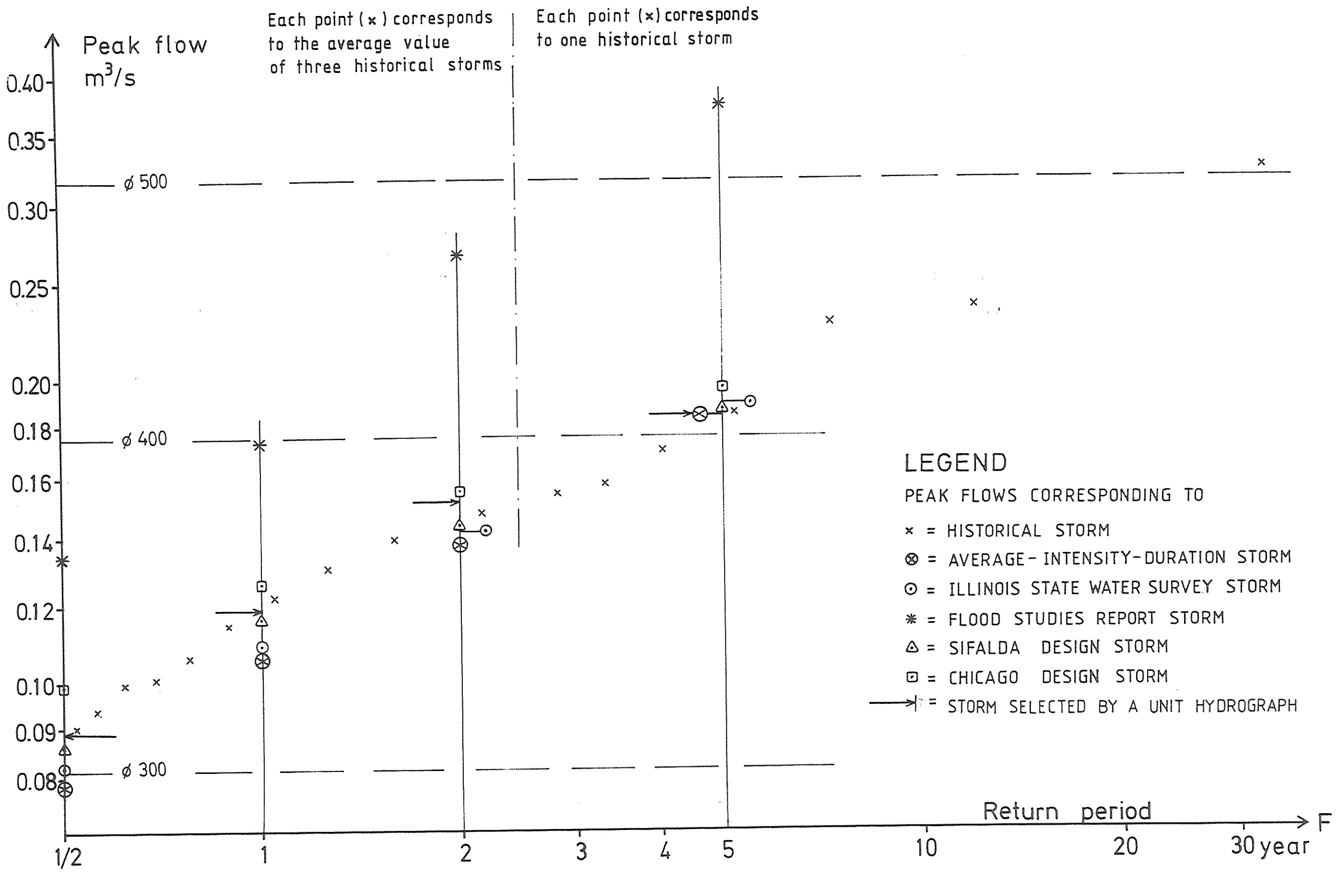
LINKÖPING 2 PIPE 7

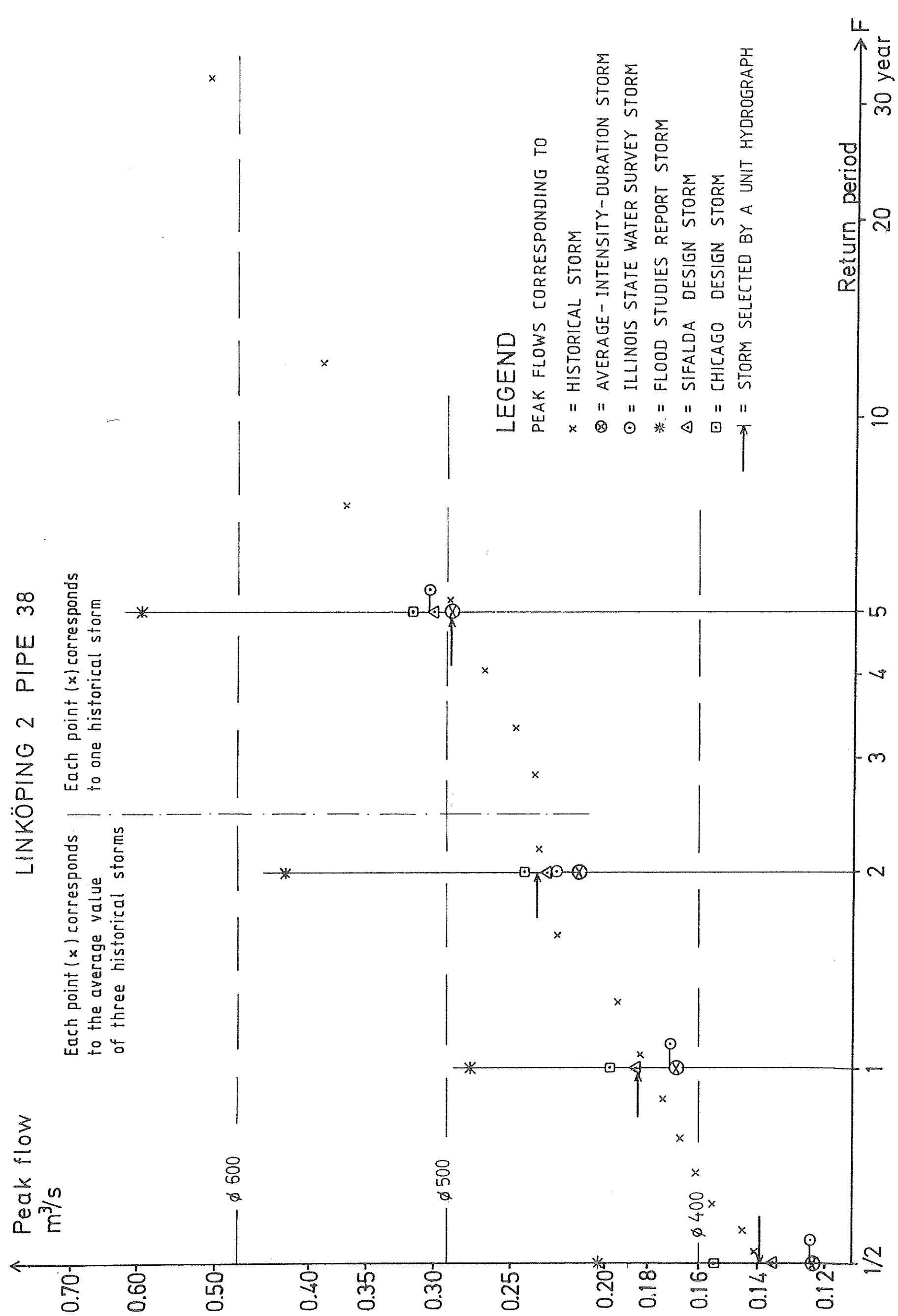


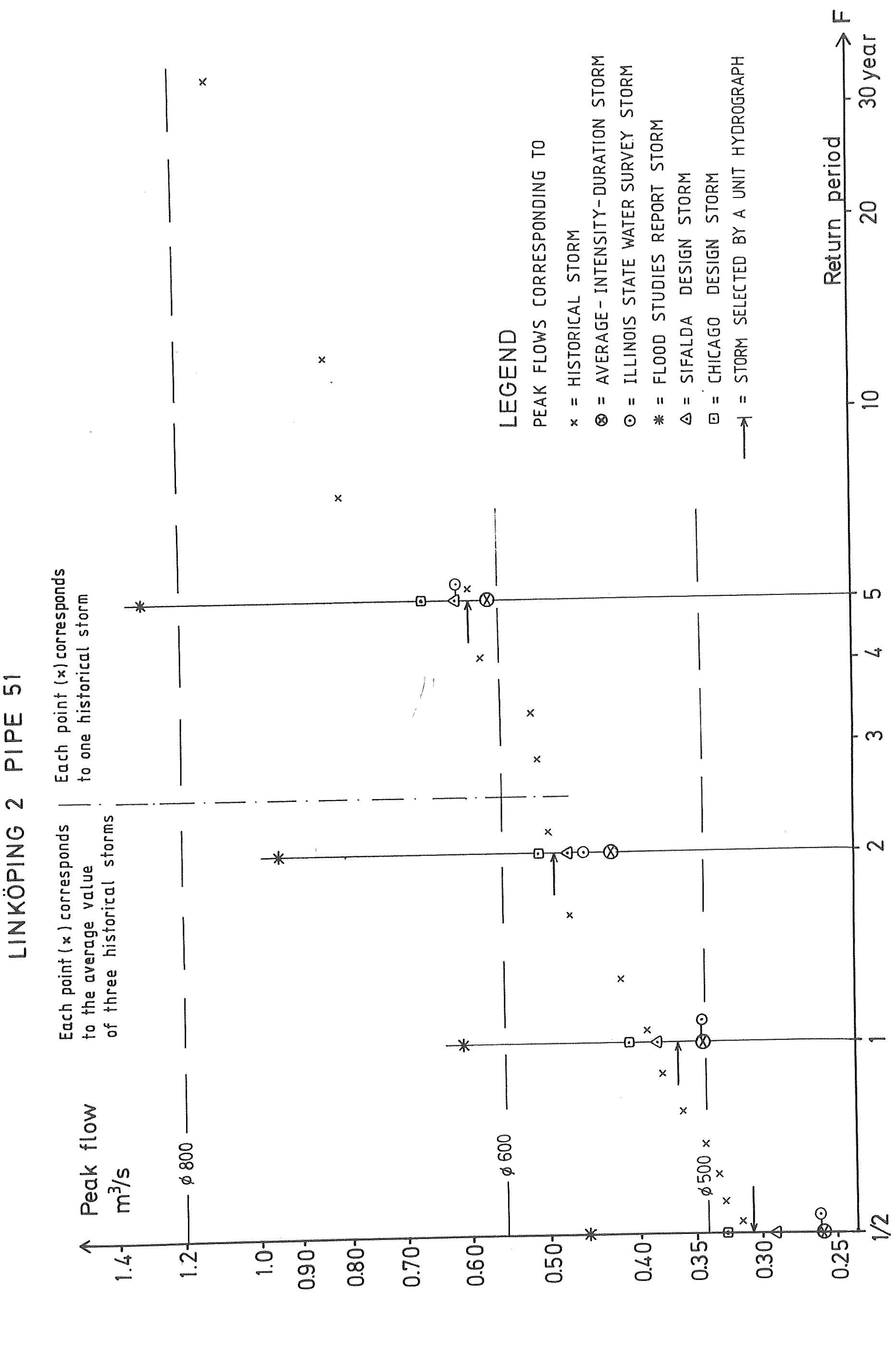

LINKÖPING 2 PIPE 15
Peak flow m³/s
0.70
0.60
0.50
0.40
0.30
0.25
0.20
0.18
0.16
0.14
ϕ 600
ϕ 500
ϕ 400
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

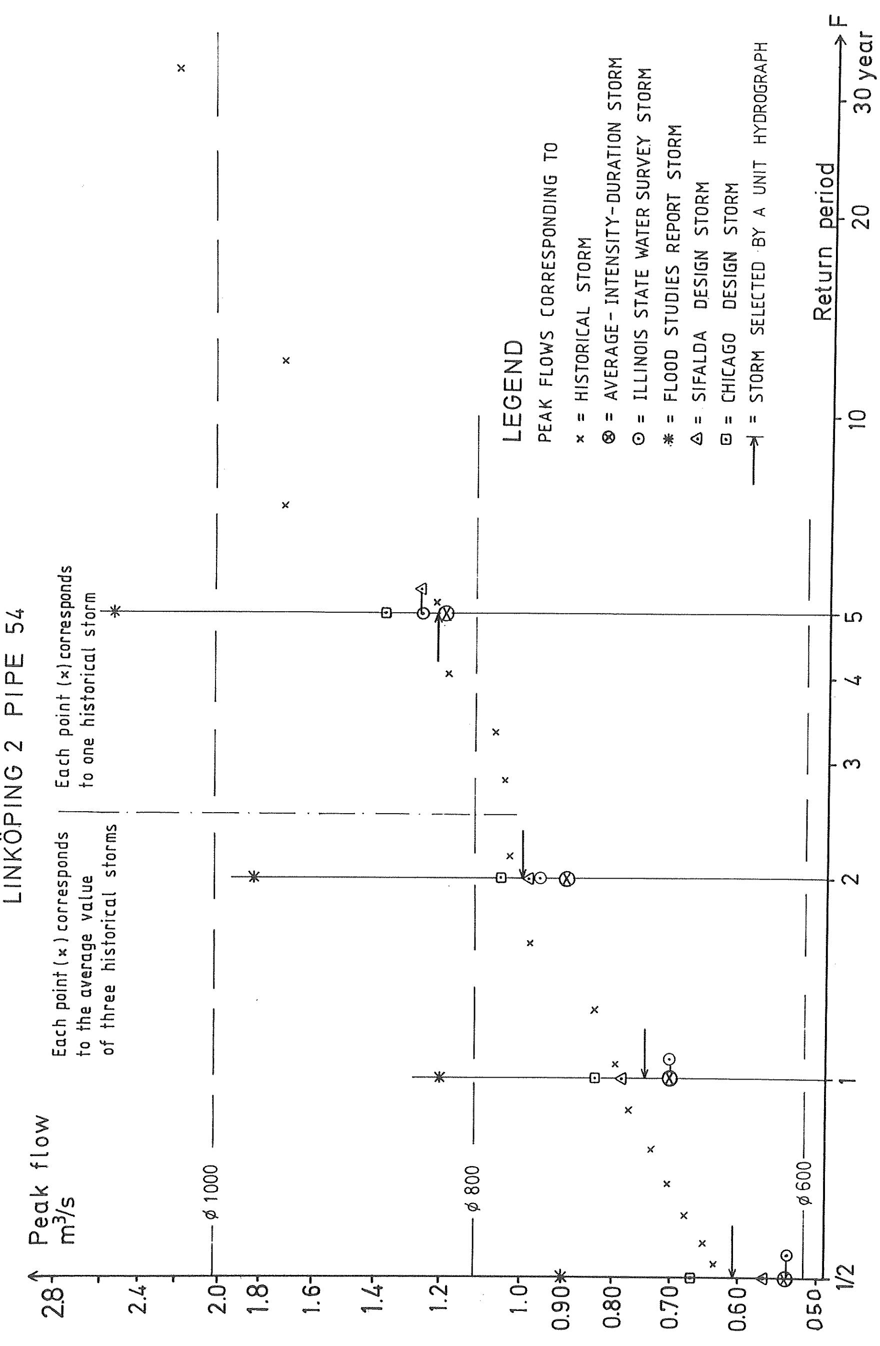

LINKÖPING 2 PIPE 29
Peak flow
m³/s
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
ø 800
ø 600
ø 500
1.2
1.0
0.90
0.80
0.70
0.60
0.50
0.40
0.35
0.30
0.25
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✱ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

LINKÖPING 2 PIPE 35




LINKÖPING 2 PIPE 38
Peak flow
m³/s
Each point (x) corresponds
to the average value
of three historical storms
Each point (x) corresponds
to one historical storm
ø 600
ø 500
ø 400
0.70
0.60
0.50
0.40
0.35
0.30
0.25
0.20
0.18
0.16
0.14
0.12
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
x = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH


LINKÖPING 2 PIPE 51
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
Peak flow m³/s
1.4
1.2
1.0
0.90
0.80
0.70
0.60
0.50
0.40
0.35
0.30
0.25
ϕ 800
ϕ 600
ϕ 500
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
x = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
✳ = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH


LINKÖPING 2 PIPE 54
Each point (×) corresponds to the average value of three historical storms
Each point (×) corresponds to one historical storm
Peak flow m³/s
2.8
2.4
2.0
1.8
1.6
1.4
1.2
1.0
0.90
0.80
0.70
0.60
0.50
ø 1000
ø 800
ø 600
1/2
1
2
3
4
5
10
20
30 year
Return period
F
LEGEND
PEAK FLOWS CORRESPONDING TO
× = HISTORICAL STORM
⊗ = AVERAGE-INTENSITY-DURATION STORM
⊙ = ILLINOIS STATE WATER SURVEY STORM
* = FLOOD STUDIES REPORT STORM
△ = SIFALDA DESIGN STORM
⊡ = CHICAGO DESIGN STORM
= STORM SELECTED BY A UNIT HYDROGRAPH

APPENDIX IV

DIFFERENCES BETWEEN THE PEAK-FLOW VALUES SIMULATED FOR DESIGN STORMS AND THE DESIGN PEAK-FLOW VALUES ESTIMATED FOR HISTORICAL STORMS

Estimation of the Deviations

The deviations in Appendix IV were obtained from the design peak-flow values given in Appendix III. The design peak-flow values for the historical storms were estimated by linear interpolation between the plotted points surrounding the considered return period. The deviations in peak flows for the different design storms are then expressed in percent of the design peak-flow values for the historical storms.

Deviations in percent of the design peak-flow values for the historical storms.

Catchment	Pipe No.	Average-Intensity-Duration Design Storm 1/2	1	2	5	Chicago Design Storm 1/2	1	2	5	Sifalda Design Storm 1/2	1	2	5
Bergsjön	09	- 4	- 3	- 4	- 7	-12	-10	-11	-15	- 8	- 8	- 8	-12
	24	- 8	- 7	- 2	+ 1	+ 2	0	+ 3	+ 5	- 4	- 3	0	+ 1
	30	-15	- 7	- 5	-10	- 8	- 2	- 3	-11	-12	- 5	- 5	-11
	42	- 1	0	+ 7	+ 4	- 6	- 4	+ 2	- 1	- 2	- 4	+ 2	- 1
	57	- 6	- 8	0	+ 2	+ 4	- 2	+ 3	+ 2	- 3	- 5	+ 1	0
	66	- 8	- 8	- 4	- 5	- 1	- 3	+ 1	0	- 8	- 8	- 1	- 2
	73	-16	-15	-12	-13	+ 2	0	+ 3	0	-11	-10	- 6	-11
Linköping 1	26	-22	-18	-15	-17	+ 4	+ 1	+ 6	+ 3	- 8	- 2	+ 3	- 5
	72	-16	-14	-14	- 5	+16	+11	+10	+18	- 3	- 1	- 2	+ 4
	73	-15	-17	-14	-12	+18	+10	+10	+12	+ 1	+ 3	+ 2	+ 1
	84	-14	-11	- 9	- 7	+10	+ 7	+ 7	+ 9	- 6	- 2	- 2	- 3
	89	-14	-14	-12	-15	+16	+12	+13	+ 7	0	+ 5	+ 5	- 5
	106	-18	-18	-16	- 8	+14	+ 7	+ 9	+12	- 3	- 1	+ 1	+ 2
	125	-14	-14	-12	-15	+17	+12	+12	+ 6	+ 1	+ 5	+ 5	- 5
Linköping 2	07	- 8	- 9	+ 1	+ 5	0	- 3	+ 4	+ 5	- 4	- 5	+ 2	+ 5
	15	- 7	- 8	- 5	- 1	+ 7	0	+ 4	+ 6	- 6	- 7	- 3	0
	29	- 7	- 7	- 8	+ 2	+ 8	+ 5	+ 2	+ 9	- 5	- 3	- 4	+ 3
	35	-11	-12	- 6	+ 1	+12	+ 5	+ 7	+ 7	- 3	- 3	- 2	+ 2
	38	-12	- 7	- 8	+ 1	+12	+ 9	+ 4	+11	- 2	+ 2	- 1	+ 5
	51	-17	-12	-13	- 4	+ 5	+ 6	+ 3	+12	- 7	- 2	- 3	+ 3
	54	-13	-11	-12	- 2	+ 7	+ 6	+ 3	+12	- 9	- 1	- 3	+ 4

Catchment	Pipe No.	Illinois State Water Survey Design Storm 1/2	1	2	5	Flood Studies Report Design Storm 1/2	1	2	5	Storm Selected by a Unit Hydrograph 1/2	1	2	5
Bergsjön	09	- 3	- 1	+ 2	- 1	+62	+72	+120	+125	- 5	- 3	- 4	+ 9
	24	- 7	- 5	+ 3	+ 7	+45	+55	+100	+115	+ 3	+ 1	+ 3	+ 2
	30	-13	- 6	- 3	- 7	+60	+75	+115	+115	- 4	+ 7	+ 2	+ 3
	42	- 3	- 2	+12	+ 9	+50	+60	+125	+135	+ 7	- 1	+10	0
	57	- 3	- 6	+ 6	+ 8	+50	+50	+100	+115	+ 4	+ 3	+ 3	+ 1
	66	- 5	- 7	+ 4	+ 2	+55	+60	+110	+120	+ 5	+ 2	+ 3	0
	73	-15	-12	- 6	- 7	+50	+55	+105	+110	+ 3	- 4	+ 3	+ 2
Linköping 1	26	-22	-17	- 8	-10	+36	+48	+ 96	+ 97	+ 2	- 2	+ 2	+ 1
	72	-15	-13	-11	0	+32	+34	+ 54	+ 72	+ 1	0	+ 4	+ 1
	73	-15	-16	-12	- 6	+32	+28	+ 47	+ 53	- 1	- 3	+ 6	+ 2
	84	-13	- 8	- 4	0	+49	+55	+ 95	+110	+ 4	+ 2	+ 6	+ 1
	89	-13	-13	- 7	-10	+30	+23	+ 37	+ 35	-13	+ 5	+ 5	+ 2
	106	-18	-18	-12	- 5	+34	+37	+ 62	+ 77	0	- 3	+ 6	+ 2
	125	-12	-13	- 5	-10	+29	+23	+ 37	+ 34	-13	+ 4	+ 5	+ 2
Linköping 2	07	- 7	- 8	+ 4	+ 7	+54	+60	+110	+130	+ 2	0	+ 1	+ 2
	15	- 6	- 7	+ 1	+ 4	+46	+43	+ 80	+ 98	+ 1	+ 2	+ 3	+ 2
	29	- 7	- 5	- 3	+ 6	+45	+49	+ 74	+105	- 2	0	- 2	+ 1
	35	- 7	- 9	- 3	+ 3	+51	+45	+ 84	+105	+ 1	- 2	+ 4	+ 2
	38	-11	- 5	- 3	+ 6	+47	+51	+ 83	+108	+ 1	+ 2	+ 2	+ 2
	51	-16	-12	- 7	+ 3	+46	+57	+ 93	+120	- 1	- 6	0	+ 1
	54	-13	-11	- 6	+ 4	+45	+52	+ 82	+110	- 3	- 6	- 2	+ 1

Deviations in percent of the design peak-flow values for the historical storms.

LIST OF FIGURES

LIST OF FIGURES continued

LIST OF FIGURES continued

LIST OF FIGURES continued

LIST OF TABLES

LIST OF TABLES continued

LIST OF TABLES continued

LIST OF TABLES continued

LIST OF TABLES continued

Appendices

LIST OF SYMBOLS

a, b, c	constants in the formula $i_m = a/(T+b)+c$
D	duration of period with maximum rainfall intensity i_{max}
d	relationship between duration of period with maximum rainfall intensity and total duration, $d = 100 \cdot D/T_{tot}$
e, f	constants in the formula $T_{3} = (e+f \cdot T) T_{tot} \cdot 1/100$ for the estimation of the duration of part 3 of the Sifalda design storm
F	return period
$G(x_1, x_2, \ldots, x_j)$	substitute for the runoff model
g, h	constants in the formula $P_{tot} = g + h \cdot \ln T$ for the estimation of the total volume of the Sifalda design storm
i	rainfall intensity
i_m	maximum average rainfall intensity during the time T
i_{max}	maximum rainfall intensity
$\bar{i}$	average rainfall intensity, $\bar{i} = P_{tot}/T_{tot}$
k, l	constants in the formula $P_{3} = (k+l \cdot T) P_{tot} \cdot 1/100$ for the estimation of the volume of part 3 of the Sifalda design storm
k_i	relationship between maximum rainfall intensity i_{max} and average rainfall intensity $\bar{i}$, $k_i = i_{max}/\bar{i}$
m	lower boundary value for the statistical exponential distribution function
N	total number of observations or values in a statistical analysis
n	number of periods for the estimation of risk
P	accumulated precipitation
P_{tot}	total rain volume during the total duration T_{tot}
P_s	rain volume before peak rainfall intensity i_{max} at time T_s

LIST OF SYMBOLS continued

$P_{①,②,③}$	volume of part ①, ②, and ③ of the Sifalda design storm
p_s	relationship between rain volume before peak rainfall intensity and total rain volume, $p_s = 100 \cdot P_s/P_{tot}$
p_t, $p_{t+\tau}$	rainfall depths at time increments t and $t+\tau$, respectively
$P(X>Q)$	probability of X being greater than the flow value Q
$P(\chi^2)$	probability of χ^2 being less than a specified value
Q	calculated peak-flow value
r	relationship between the time prior to peak rainfall intensity (T_f) and the total duration (T) in the Chicago design storm
r_s	correlation coefficient of the correlation between the duration of the maximum average intensity period and the volumes and durations of the different parts of the Sifalda design storm
r_τ	correlation between rainfall depths at time t and $t+\tau$, respectively
T	duration of maximum average rainfall intensity period
T_B	time to beginning of period with maximum rainfall intensity i_{max}
T_e	time from peak rainfall intensity to the end of rainfall in the Chicago design storm
T_f	time from peak rainfall intensity to the start of rainfall in the Chicago design storm
T_s	time to peak rainfall intensity
T_{tot}	total duration
t_B	relationship between time to beginning of period with maximum rainfall intensity and total duration, $t_B = 100 \cdot T_B/T_{tot}$
t_e	time counted from peak intensity towards the end of rainfall in the Chicago design storm

LIST OF SYMBOLS continued

t_f	time counted from peak intensity towards the start of rainfall in the Chicago design storm
t_s	relationship between time to peak rainfall intensity and total duration, $t_s = 100 \cdot T_s/T_{tot}$
Δt	length of time step
u, v	constants in the formula $T_{tot} = u + v \cdot \ln T$ for the estimation of the total duration of the Sifalda design storm
$x_1, x_2, \ldots, x_j$	parameters in a runoff model
y_i	plotting position for observation number i in increasing order, $y_i = \ln F$
z	slope of the statistical exponential distribution function
η	coefficient for compensation of model error
λ	ratio between simulated and observed runoff values (volumes or peaks)
σ	standard deviation
χ^2	statistical test quantity
τ	time distance between the time t and $t+\tau$
Ω	coefficient of variation or the standard deviation divided by the mean value for a parameter

REFERENCES

Arnell, V. (1974): Intensity-Duration-Frequency Relationships for Heavy Rainfalls in Göteborg during the 45 Year Period 1926-1971. Chalmers University of Technology, Urban Geohydrology Research Group, Report No. 5, Göteborg (in Swedish).

Arnell, V. (1978): Analysis of Rainfall Data for Use in Design of Storm Sewer Systems. Proc. of the International Conference on "Urban Storm Drainage" held at the University of Southampton, April 1978, Pentech Press, London.

Arnell, V. (1980): Description and Validation of the CTH-Urban Runoff Model. Chalmers University of Technology, Department of Hydraulics, Report Series A:5, Göteborg.

Arnell, V.; Asp, T. (1979): The Duration and Volume of Precipitation at Lundby, Göteborg, 1921-1939. Chalmers University of Technology, Urban Geohydrology Research Group, Report No. 44, Göteborg (in Swedish).

Arnell, V.; Dahlström, B; Falk, J; Niemczynowicz, J. (1980): Rainfall in Urban Areas. State of the Knowledge and Research Plan. Swedish Council for Building Research, Report G15:1980, Stockholm (in Swedish).

Arnell, V.; Strandner, H.; Svensson, G. (1980): Storm-Water Runoff, Quantity and Quality at Ryd, Linköping, 1976-1977. Chalmers University of Technology, Urban Geohydrology Research Group, Report No. 48, Göteborg (in Swedish).

Bandyopadhyay, M. (1972): Synthetic Storm Pattern and Run-off for Gauhati, India. Journal of the Hydraulics Division, ASCE, Vol. 98, No. HY5, Proc. Paper 8887, pp 845-857.

Dahlström, B. (1979): Regional Distribution of Rainfall Intensity - A Climatic Analysis. Swedish Council for Building Research, Report R18:1979, Stockholm (in Swedish).

Desbordes, M. (1978): Urban Runoff and Design Storm Modeling. Proc. of the International Conference on "Urban Storm Drainage" held at the University of Southampton, April 1978, Pentech Press, London.

DIF, Danish Society of Engineers (1974): Evaluation of Intensity-Duration-Frequency Curves. Sewage Water Committe, Report No. 16, Teknisk Forlag A/S, Köpenhamn (in Danish).

Folland, C.K. (1978): Meteorological Aspects of Drainage Design. Meteorological Office, Rainfall Memorandum No. 3, Revised edition, March 1978, London.

Folland, C.K.; Colgate, M.G. (1978): Recent and Planned Rainfall Studies in the Meteorological Office with an Application to Urban Drainage Design. Urban Storm Drainage, Proc. of the International Conference held at the University of Southampton, April 1978, Pentech Press, London.

Grace, R.A.; Eagleson, P.S. (1967): A Model for Generating Synthetic Sequences of Short-Time-Interval Rainfall Depths. Proc. of "The International Hydrology Symposium", September 6-8, 1967, Colorado State University, Fort Collins, Colorado.

Gustavsberg (1975): Pipe Book. Distributed by Scandiaconsult, Fack, S-102 60 Stockholm. Stockholm (in Swedish).

Howard, C.D.D. (1976): Theory of Storage and Treatment - Plant Overflows. Journal of the Environmental Engineering Div., ASCE, EE4, August 1976.

Huff, F.A. (1967): Time Distribution of Rainfall in Heavy Storms. Water Resources Research, Vol. 3, No. 4, pp 1007-1019.

IMSL (1975): International Mathematical and Statistical Libraries Inc. IMSL, Library 1, Fortran IV, S/370-360, Houston, Texas.

Johansen, L. (1979): Design Rainfalls for Sewer Systems. Report 79-2, Department of Sanitary Engineering, Technical University of Denmark, Köpenhamn (in Danish).

Johansen, L.; Harremoës, P. (1979): The Use of Historical Storms for Urban Drainage Design. International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky.

Johansen, N.B. (1981): Method for Computation of Overflow Volumes and Load of Pollutants. Nordic Seminar on Pollutants in Storm Water, The Technical University of Norway, August 24-25, 1981, Trondheim.

Keers, J.F. (1977): Rainfall Criteria for Urban Drainage Design. Meteorological Magazine, 106, pp 117-126.

Keers, J.F.; Wescott, P. (1977): A Computer - Based Model for Design Rainfall in the United Kingdom. Meteorological Office, Scientific Paper, No. 36, Her Majesty's Stationery Office, London.

Keifer, C.J.; Chu, H.H. (1957): Synthetic Storm Pattern for Drainage Design. Journals of the Hydraulics Div., ASCE, Vol. 83, No. HY4, August 1957. Discussion by McPherson in Vol. 84, No. HY1, Feb. 1958.

Kidd, C.H.R.; Packman, J.C. (1980): Selection of Design Storm and Antecedent Condition for Urban Drainage Design. Institute of Hydrology, Report No. 61, Wallingford, Oxon.

Kite, G.W. (1977): Frequency and Risk Analyses in Hydrology. Water Resources Publications, Fort Collins, Colorado.

Lindholm, O. (1974): A Pollutional Analysis of the Combined Sewer System. Division of Hydraulic Engineering, University of Trondheim, Norwegian Institute of Technology, Trondheim.

Lindholm, O. (1975): Selection of Storm for Modeling. Committe for Treatment of Sewage Water, Pra 6, Oslo (in Norwegian).

Lyngfelt, S. (1978): An Analysis of Parameters in a Kinematic Wave Model of Overland Flow in Urban Areas. Department of Hydraulics, Chalmers University of Technology, Report Series B:13, Göteborg.

Lyngfelt, S. (1981): Design of Storm Sewer Systems, The Rational Method. Chalmers University of Technology, Urban Geohydrology Research Group, Report No. 56, Göteborg (in Swedish).

Marsalek, J. (1977): Runoff Control on Urbanizing Catchments. Symposium on the "Effects of Urbanization and Industrialization on the Hydrological Regime and on Water Quality" in Amsterdam, October 1977, IAHS-AISH Publication No. 123.

Marsalek, J. (1978a): Synthesized and Historical Storms for Urban Drainage Design. Paper presented at "International Conference on Urban Storm Drainage" April 1978 at University of Southampton, England, Pentech Press, London.

Marsalek, J, (1978b): Research on the Design Storm Concept. ASCE Urban Water Resources Research Program, Technical Memorandum No. 33, New York.

McPherson, M.B. (1977): The Design Storm Concept. Urban Runoff Control Planning. A Study of ASCE, Urban Water Resources Research Council, New-York.

Natural Environment Research Council (1975): Flood Studies Report, Vol. 2, Meteorological Studies. Natural Environment Research Council, London.

Nilsdal, J.-A. (1981): Technical-Economic Design of Sewers, A Literature Review of Computer Models. Department of Hydraulics, Chalmers University of Technology, Report Series B:27,Göteborg (in Swedish).

Packman, J.C.; Kidd, C.H.R. (1980): A Logical Approach to the Design Storm Concept. Water Resources Research, Vol. 16, No. 6, pp 994-1000, December 1980.

Patry, G.; McPherson, M.B. (1979): The Design Storm Concept. Proc. of a Seminar at Ecole Polytechnique de Montreal and of a related Session of the American Geophysical Union, Urban Water Resources Research Group, Ecole Polytechnique de Montreal, Civil Engineering Department, GREMU - 79/2, Montreal.

Preul, H.; Papadakis, C.N. (1973): Development of Design Storm Hyetographs for Cincinnati, Ohio. Water Resources Bulletin, Vol. 9, No. 2, April 1973.

Proctor and Redfern Ltd. and James F MacLaren Ltd. (1976): Storm Water Management Model Study Vol. 1. Research Report No. 47, Research Program for the Abatement of Municipal Pollution under Provision of the Canada-Ontario Agreement on Great Lakes Water Quality, Toronto.

Siegel, S. (1956): Nonparametric Statistics for the Behavioral Sciences. McGraw-Hill Book Comp., New-York.

Sieker, F (1978): Investigation of the Accuracy of the Postulate "Total Rainfall Frequency Equal Flood Peak Frequency". Proc. of the International Conference on "Urban Storm Drainage" held at the University of Southampton, April 1978. Pentech Press, London.

Sifalda, V. (1973): Entwicklung eines Berechnungsregens für die Bemessung von Kanalnetzen. gwf - WASSER/ ABWASSER 114 (1973) H9.

Thorndal, U. (1971): Precipitation Hydrographs. Stads og havneingeniören No. 7, Köpenhamn (in Danish).

Tang, W.H.; Mays, L.W.; Yen, B.C. (1975): Optimal Risk - Based Design of Storm Sewer Networks. Journal of the Environmental Eng. Div., ASCE, Vol. 101, No. EE 3, June 1975.

Terstriep, M.; Stall, J.B. (1974): The Illinois Urban Drainage Area Simulator. Illinois State Water Survey, Bulletin 58, Urbana, Ill.

Transport and Road Research Laboratory. Department of the Environment (1976): A Guide for Engineers to the Design of Storm Sewer Systems. Road Note 35, Second Ed., London.

Tung, Y.-K.; Mays, L.W. (1980): Risk Analysis for Hydraulic Design. Journal of the Hydraulics Div., ASCE, Vol. 106, No. HY5, May 1980, pp 893-913.

Urbonas, B. (1979): Reliability of Design Storms in Modeling. International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky.

VAV, Swedish Water and Waste Water Works Association (1976): Manual for Design of Sewer Pipes. VAV, Publication P28, Stockholm (in Swedish).

Walesh, S.G.; Lau, D.H.; Liebman, M.D. (1979): Statistically-Based Use of Event Models. International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky.

Watkins, L.H. (1962): The Design of Urban Sewer Systems. Road Research Technical Paper No. 55, Dept. of Scientific and Industrial Research, London.

Wenzel, H.G.; Voorhees, M.L. (1978): Evaluation of the Design Storm Concept. Paper presented at the 1978 Fall Meeting of AGU, December 1978, San Francisco.

Wenzel, H.G.; Voorhees, M.L. (1979): Sensitivity of Design Storm Frequency. In: "The Design Storm Concept", Patry, G.; McPherson, M.B. (Editors), Proceedings of a Seminar at École Polytechnique and of a Related Session of the American Geophysical Union, Urban Water Resources Research Group, École Polytechnique de Montreal, Civil Engineering Department, GREMU-79/2, Montreal.

Vogel, J.L.; Huff, F.A. (1977): Heavy Rainfall Relations over Chicago and Northeastern Illinois. Water Resources Bulletin, Vol. 13, No. 5, October 1977.

Yen, B.C.; Wenzel, H.G.; Mays, L.W.; Tang, W.H. (1976): Advanced Methodologies for Design of Storm Sewer Systems. University of Illinois at Urbana - Champaign, Water Resources Center, Research Report No. 112, Urbana.

Yevjevich, V. (1972): Stochastic Processes in Hydrology. Water Resources Publications, Fort Collins, Colorado.

Department of Hydraulics
Chalmers University of Technology

Report Series A

A:1 Bergdahl, L.: Physics of ice and snow as affects thermal pressure. 1977.

A:2 Bergdahl, L.: Thermal ice pressure in lake ice covers. 1978.

A:3 Häggström, S.: Surface Discharge of Cooling Water. Effects of Distortion in Model Investigations. 1978.

A:4 Sellgren, A.: Slurry Transportation of Ores and Industrial Minerals in a Vertical Pipe by Centrifugal Pumps. 1978.

A:5 Arnell, V.: Description and Validation of the CTH-Urban Runoff Model. 1980.

A:6 Sjöberg, A.: Calculation of Unsteady Flows in Regulated Rivers and Storm Sewer Systems. (in Swedish). 1976.

A:7 Svensson, T.: Water Exchange and Mixing in Fjords. Mathematical Models and Field Studies in the Byfjord. 1980.

Report Series B

B:1 Bergdahl, L.: Beräkning av vågkrafter. 1977. Ersatts med 1979:07

B:2 Arnell, V.: Studier av amerikansk dagvattenteknik. 1977.

B:3 Sellgren, A.: Hydraulic Hoisting of Crushed Ores. A feasibility study and pilot-plant investigation on coarse iron ore transportation by centrifugal pumps. 1977.

B:4 Ringesten, B.: Energi ur havsströmmar. 1977.

B:5 Sjöberg, A. and Asp, T.: Brukar-anvisning för ROUTE-S. En matematisk modell för beräkning av icke-stationära flöden i floder och kanaler vid strömmande tillstånd. 1977.

B:6 Annual Report 76/77.

B:7 Bergdahl, L. and Wernersson, L.: Calculated and expected Thermal Ice Pressures in Five Swedish Lakes. 1977.

B:8 Göransson, C-G. and Svensson, T.: Drogue Tracking - Measuring Principles and Data Handling.

B:9 Göransson, C-G.: Mathematical Model of Sewage Discharge into confined, stratified Basins - Especially Fjords.

B:10 Arnell, V. and Lyngfelt, S.: Beräkning av dagvattenavrinning från urbana områden. 1978.

B:11 Arnell, V.: Analysis of Rainfall Data for Use in Design of Storm Sewer Systems. 1978.

B:12 Sjöberg, A.: On Models to be used in Sweden for Detailed Design and Analysis of Storm Drainage Systems. 1978.

B:13 Lyngfelt, S.: An Analysis of Parameters in a Kinematic Wave Model of Overland Flow in Urban Areas. 1978.

B:14 Sjöberg, A. and Lundgren, J.: Manual for ILLUDAS (Version S2). Ett datorprogram för dimensionering och analys av dagvattensystem.

B:15 Annual Report 78/79.

B:16 Nilsdal, J-A. and Sjöberg, A.: Dimensionerande regn vid höga vattenstånd i Göta Älv.

B:17 Stöllman, L-E.: Närkes Svartå. Hydrologisk inventering. 1979.

B:18 Svensson, T.: Tracer Measurements of Mixing in the Deep Water of a Small, Stratified Sill Fjord.

B:19 Svensson, T., Degerman, E., Jansson,B. and Westerlund, S.: Energiutvinning ur sjö- och havssediment. En
R76:1980 förstudie.

B:20 Annual Report 1979

B:21 Stöllman, L-E.: Närkes Svartå. Inventering av vattentillgång och vattenanvändning. 1980.

B:22 Häggström, S. och Sjöberg, A.: Effects of Distortion in Physical Models of Cooling Water Discharge. 1979.

B:23 Sellgren, A.: A Model for Calculating the Pumping Cost of Industrial Slurries. 1981.

B:24 Lindahl, J.: Rörelseekvationen för en kabel. 1981

B:25 Bergdahl, L. och Olsson, G.: Konstruktioner i havet. Vågkrafter, rörelser. En inventering av datorprogram.

B:26 Annual Report 1980.

B:27 Nilsdal, J-A.: Teknisk-ekonomisk dimensionering av avloppsledningar. En litteraturstudie om datormodeller. 1981.

B:28 Sjöberg, A.: The Sewer Network Models DAGVL-A and DAGVL-DIFF. 1981.

B:29 Moberg, G.: Anläggningar för oljeutvinning till havs. Konstruktionstyper, dimensioneringskriterier och positioneringssystem. 1981

B:30 Sjöberg, A. och Bergdahl, L.: Förankringar och förankringskrafter. 1981