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GEOHYDROLOGISKA FORSKNINGSGRUPPEN

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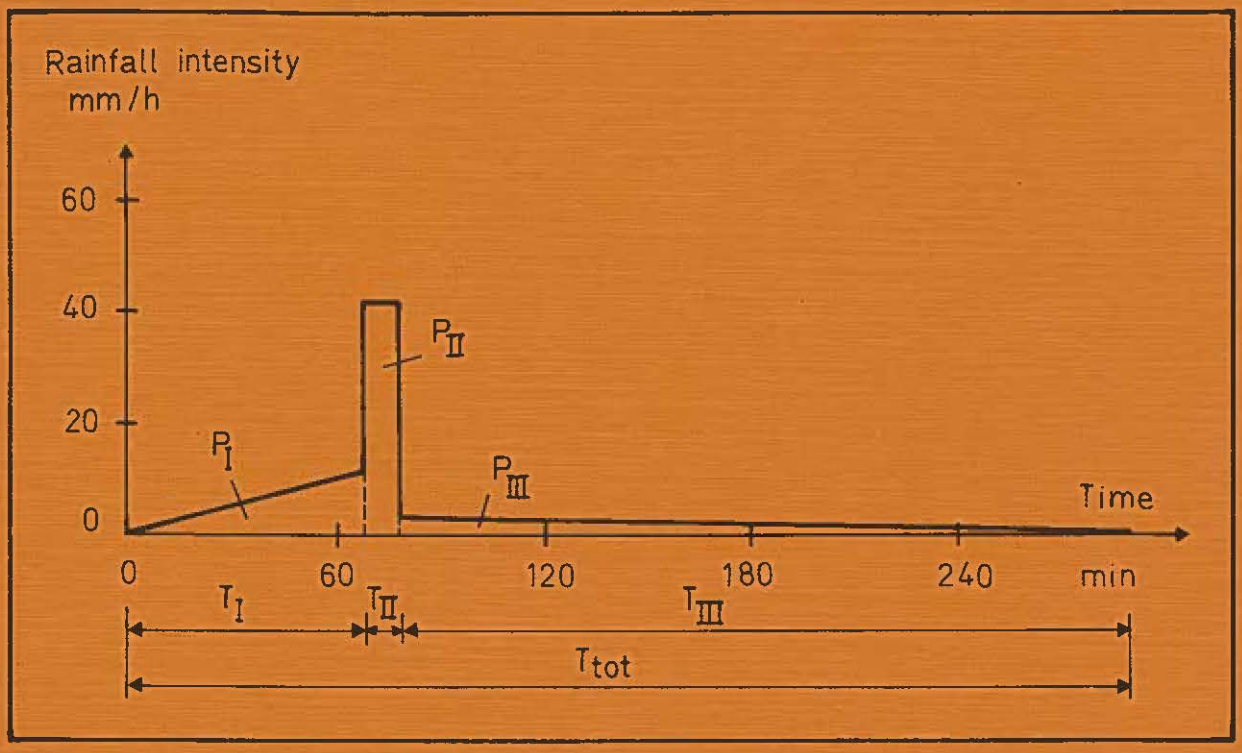

Viktor Arnell
Henriette Melin

Meddelande nr 76

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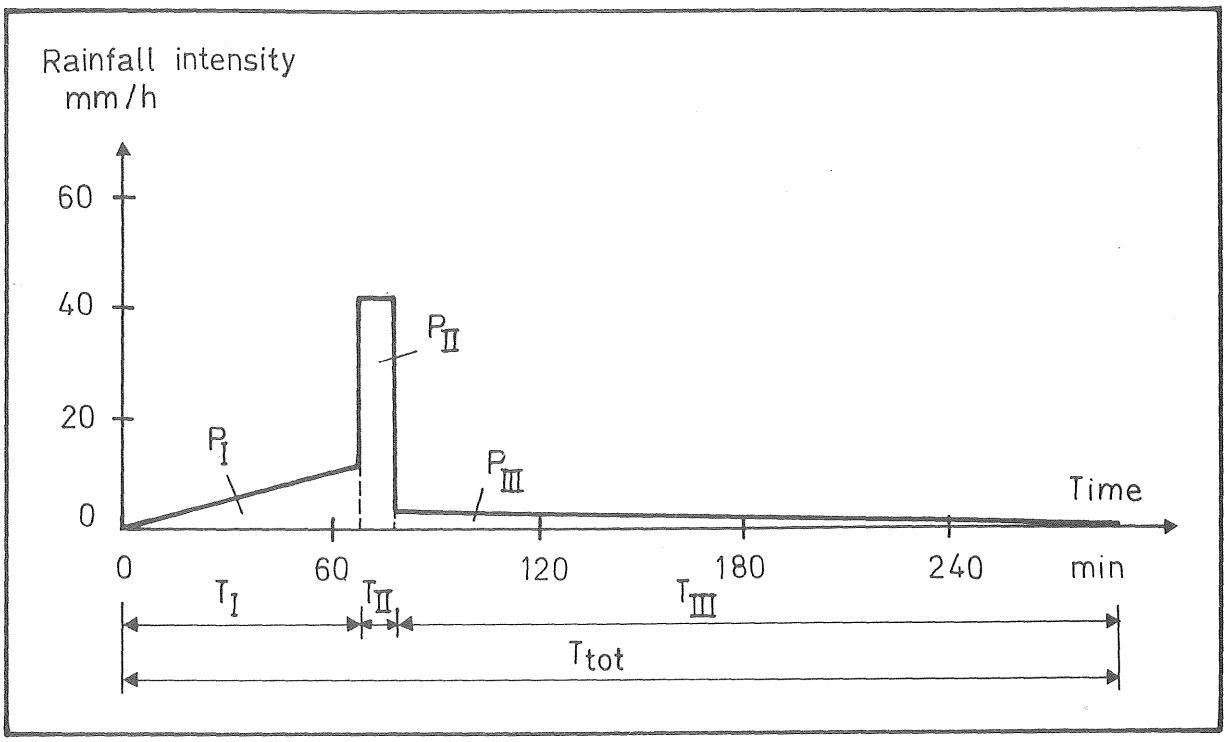

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PREFACE

This report includes the result of research on rainfall data for design and analysis of detention basins. Earlier reports on rainfall data deal with the evaluation of new Intensity-Duration-Frequency curves for Göteborg, rainfall statistics on durations and volumes for different rainfall intensity levels, and rainfall data for design of sewer pipe systems with computer models. After the present study the research continues with studies of rainfall data for the design of detention basins aimed to reduce overflow volumes.

The manuscript was typed by Ann-Marie Hellgren and the figures were drawn by Alicja Janiszewska.

Programming, preparation of rainfall data and computer simulations were made by Henriette Melin.

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Linköping June, 1984

Viktor Arnell

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SUMMARY

This report describes the selection of rainfall data for design of detention basins. The use of design storms (the uniform intensity design storm, the Chicago design storm and the Sifalda design storm) were compared with the use of historical storms where the historical storms were assumed to give the most correct result.

Historical storms were selected from an 18-year rainfall record and local coefficients for the design storms were estimated from the same record.

The historical storms as well as the design storms were used for design of detention basins located at the outlet in each of three test catchments of the sizes 0.154 km^2, 1.45 km^2, and 0.185 km^2. The detention basins were emptied through outlets of nozzle-type located at the bottom. Two permitted maximum water depths, namely 2.0 and 3.5 m, were tested and the permitted maximum outflows were varied between 5 and 30 l/s.ha counted for the contributing surfaces only.

For the runoff simulations the CTH-Urban Runoff Model was used, which includes the processes of infiltration, surface depression storage, overland flow, gutter flow, and pipe flow. A special sub-model simulated the flow through the detention basins and designed the size of the horizontal area of the basins so the maximum water depth was not exceeded. The outflow of each basin was a function of the water depth.

The result showed that it is important to simulate the inflow hydrograph to the basins and that the design storm has a correct total rainfall volume. The resulting basin volumes were not much influenced by the maximum water depth. The uniform intensity design storm underestimated the basin volumes by on average 18% and should not be used for practical applications. The use of the Chicago design storm resulted in underestimated basin volumes of on average 9% and the Sifalda design storm caused overestimated volumes of on average 13%.

A manual method recommended by the Swedish Water and Waste Water Works Association (VAV P31) were also tested. The inflows to the basins were calculated by a linear time-area method and the time of concentration by an equation taking into account the area of the catchment, the length and slope of the main pipe, and the rainfall intensity. The design rainfall intensities were taken from the intensity-duration-frequency curves. The design outflows from the basins were obtained as the mean values of the outflows when the basins were full and empty.

The way the outflow is obtained was found important in the manual method. The method gave underestimated basin volumes of 5-20%. The underestimations were larger in the large catchment than in the two smaller catchments. Also, the underestimations were larger for the small outflows than for the larger outflows. To compensate for the underestimations when using the manual method the basin volumes should be increased by 5-20% depending on the total size of the contributing areas and the design outflow from the basin.

1 INTRODUCTION

Detention basins in sewer systems are used to reduce the runoff peak flows downstream of the basins or to control the amount of overflow of untreated sewage water to the recipient, or to reduce the amount of pollution in the sewage water.

Rainfall data for design of the size of a basin must be selected to meet the aim of the basin. As a general rule basins are designed to store a specified volume of water while sewer pipes are designed to carry peak flows. The same rainfall data may not be appropriate to use for the design of basins as for the design of pipes. Other factors influencing the selection of rainfall data are if the basin is located off-line or on-line. Off-line basins are located separately from the sewer system and the water enters the basin via, for example, a weir. On-line basins are a part of the hydraulic flow system where the water is flowing through the basin and is detained by controling the outflow from the basin by, for example, a nozzle, a weir, or a pump. This report describes the selection of rainfall data for design of on-line basins with an outlet of a nozzle-type located at the bottom. The permitted maximum outflow and the permitted maximum water depth in the basin are limited and flooding is not permitted more frequently than stated by the design return period used.

Two different kinds of rainfall data can be used for the design of detention basins:

* Design storms estimated from Intensity-Duration-Frequency (I-D-F) 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 historical storms give more correct results for all types of areas, and non-linear effects in the runoff process that influence the flow statistics can be taken into account. Historical storms can be more expensive and complicated to use, but that problem can be minimized through well-developed manuals and computer routines.

A number of earlier investigations about detention basins are reported in the literature but only a few deal with the selection of rainfall data for design of the basins. Johansen (1979) found that the use of historical rainfalls, combined with a unit-hydrograph to the basin, gave approximately the same sizes of the basins as by using the constant rainfall intensity obtained from the I-D-F-relationships as direct inflow to the basin. For outflows less than 10 l/s ha_{red} the use of the I-D-F curves gave a small underestimation and for outflows larger than 10 l/s ha_{red} the use of the I-D-F curves gave overestimations. The outflow was assumed to be constant as a function of time. The use of the historical rainfalls as direct inflow to the basins gave overestimated volumes compared to the volumes obtained by using the I-D-F curves. Thus, it was found important to use some sort of runoff model to calculate the inflow to the basin.

Stahre (1981) found that the use of a model like ILLUDAS (see Terstriep and Stall, 1974) gave smaller basins than the use of a linear time-area diagram when using the same rainfall data in the

form of a constant rainfall intensity obtained from the I-D-F curve. The use of a design storm presented by Sifalda (1973) and shown in Fig. 2.5 gave much larger volumes of the basins than the I-D-F rainfall data. This could be expected since the Sifalda design storm includes the I-D-F design storm and with rainfall added prior to and after this storm. Stahre found it important to specify the outflow correctly, for example if the given outflow is the maximum permitted peak outflow or the maximum permitted average outflow.

Marsalek (1978) obtained larger outflows than the design outflows when routing the Chicago design storm through basins designed by using historical storms. He explained that by the Chicago design storm caused larger runoff volumes than the historical storms did.

Pecher (1978) found that the use of historical rainfalls and a unit-hydrograph method combined with a detention basin model (function not specified) resulted in smaller basin volumes than conventional design of retention basins using the I-D-F relationships.

Different studies on design of detention basins have also been reported by Colyer and Wooldridge (1979), Colyer (1980), Wooldridge (1981), and Donahue et al. (1981), but in none of these last references have comparisons of design of detention basins for different types of rainfall data been made.

In the present report a definition of independent flooding events and the criteria for design of the basins is first given. The precipitation data used in the form of an 18-year continuous historical record and the design storms derived from the record are described. For the runoff simulations the CTH-Model (Arnell, 1980) are used. The detention basin model calculates the outflows as a function of the water depths in the basins and determine the areas of the basins given the maximum water depths and the maximum outflows. Three test catchments are used and detention basins are located at the outlet of each basin. The detention basins are designed both by using historical storms and by using different design storms. Traditional design by using the I-D-F curves and

linear time-area diagrams are carried out for comparison. The results of the different designs are compared assuming the use of historical storms combined with the CTH-Model giving the most correct designs. Recommendations are given for practical applications.

2 REALIZATION OF THE DESIGN OF DETENTION BASINS

2.1 Definition of an Independent Flooding Event and Design Criteria

Independent runoff events must be defined to make possible a statistical analysis of basin volumes calculated for the historical storms. The basins are designed using the same flooding frequencies as for the surrounding sewer pipes. Arnell (1982) found by using autocorrelation analysis the necessary time between independent rainfall intensity values to be 4 hours and he used that as the necessary rain-free time period for separation of independent rainfall events used for design of sewer pipes. The same definition is used in the present investigation combined with the condition that the detention basins must be emptied between two rainfall events otherwise the events must be considered as one event.

The detention basins are designed under the constraints of a permitted maximum peak outflow varied between 5 and 30 $l/s \cdot ha_{red}$ (ha_{red} = size of the areas contributing to runoff), and a permitted maximum water depth in the basin varied between 2 and 3.5 m. A specified baseflow for self-cleaning, given in proportion of the peak outflow, is allowed to pass the basins before the storage start. The flooding frequencies are varied between six months and 5 years.

2.2 Rainfall Data Used in the Investigation

In the investigation historical rainfall data has been used from Lundby, Göteborg, for the period 1921-1939 making an 18-year (1922 excluded because of missing data) continuous rainfall series. The time increments vary because only the break-points were registered when the rainfall curves were transformed to the computer. For the runoff simulations by the CTH-Model the data of interest were transformed to hyetographs with a time step of one minute and divided into individual events by using the result of the earlier mentioned autocorrelation analysis.

A rainfall event was defined as a series of rainfall intensity values where:

(1) the intensity values ≧ 0.1 mm/h
(2) intensity values < 0.1 mm/h were allowed during time intervals of maximum 4 hours within the rainfall
(3) the total duration of the rainfall ≧ 2 minutes
(4) the total volume of the rainfall ≧ 0.5 mm.

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. Further details concerning the rainfall record are given by Arnell (1982).

To reduce the computer costs a screening of the rainfall events was carried out to find the events necessary to obtain a correct statistical determination of the basin volumes. The screening was carried out by calculating a preliminary basin volume for each rainfall and the rainfalls that caused the largest volumes calculated by the method described below were selected for the final investigation.

The preliminary "detention basin volumes" were calculated by considering the rainfall as the direct input to the basin and assuming a constant outflow if there was water stored in the basin, otherwise the outflow would be equal to the inflow. The storage equation was expressed by the numerical difference scheme:

$$\frac{M^{j+1}-M^{j}}{\Delta t} = Q_1^{j+1} - Q_2^{c} \; ; \quad M^{j+1}, M^{j} \geqq 0 \qquad (2.1)$$

where

M^{j+1}, M^{j} = volume of water in the basin
Q_1 = inflow to the basin
Q_2^c = constant outflow from the basin when water is stored
j, j+1 = time j and j+1
Δt = length of time step between times j and j+1.

When the basin was empty:

$$Q_2 = Q_1^{j+1} \qquad (2.2)$$

The method used for screening of the continuous rainfall series resulted in a reduced number of rainfalls which include all events necessary to make a correct statistical analysis of the basin volumes and it also includes a safety-margin since the value of the constant outflow Q_2^c was chosen small enough. Johansen (1979) used the method above for direct design of the basins and found that it gave larger basin values than the use of historical storms and a unit hydrograph method for calculation of the inflows.

The value of Q_2^c was varied between 1.0 $l/s \cdot ha_{red}$ and 30 $l/s \cdot ha_{red}$. For each outflow the rainfalls corresponding to the 100 largest basin volumes were identified. Since many rainfalls were identical for the different outflows a total number of 176 rainfalls were selected from the total number of about 2300 events. Data concerning the events are given in Table 2.1.

Designs of the detention basins have been done for three different types of design storms:

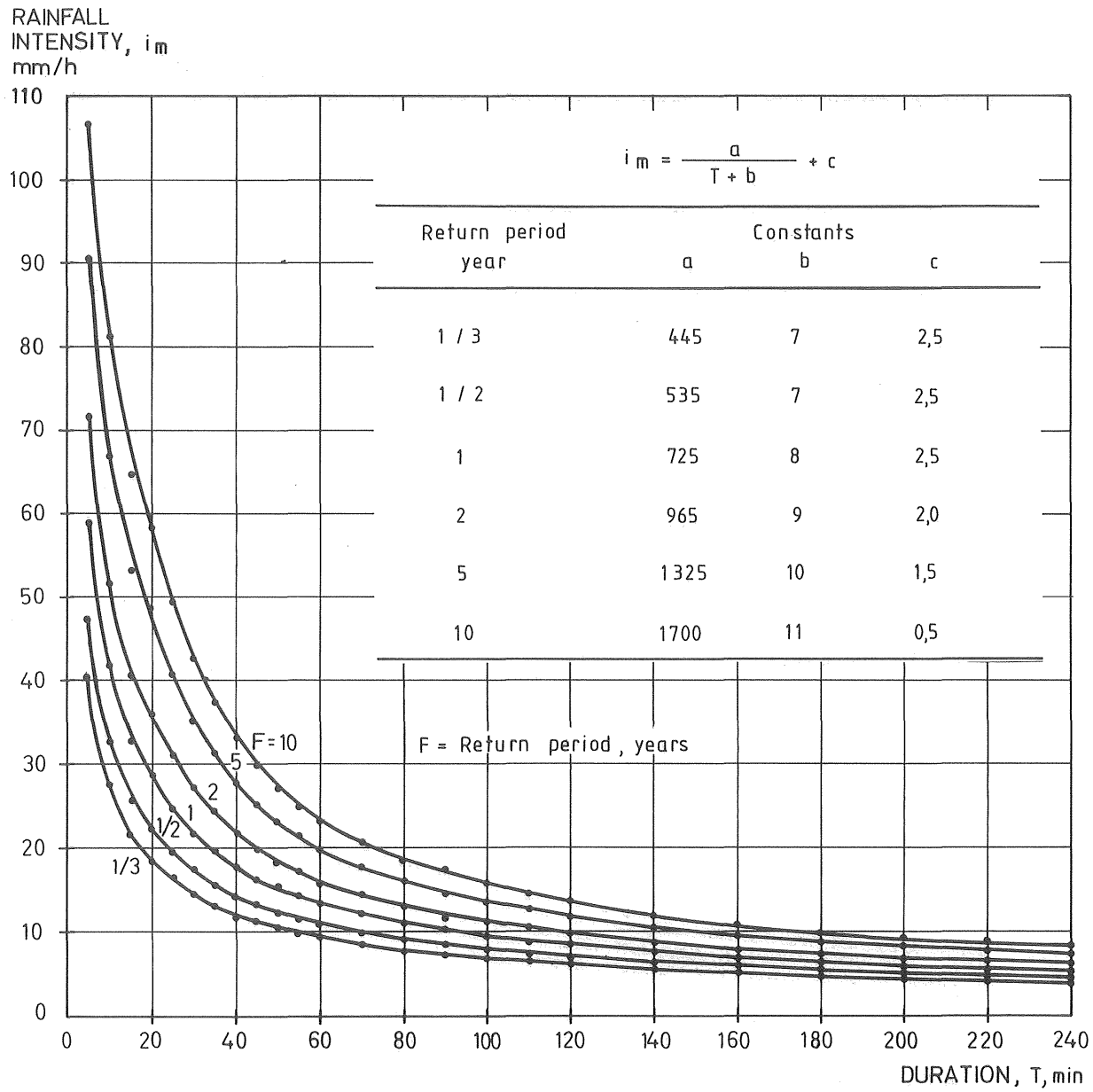
(1) Average-Intensity-Duration (I-D-F) design storm
(2) Chicago design storm
(3) Sifalda design storm.

All these design storms are in one way or another connected with the I-D-F relationships. Arnell (1982) has presented I-D-F curves (see Fig. 2.1) and the following mathematical expressions fitted to the curves:

$$i_m = \frac{a}{T+b} + c \qquad (2.3)$$

where

i_m = maximum average intensity during the duration T
a, b, c = constants listed in Fig. 2.1.



$$i_m = \frac{a}{T + b} + c$$

Return period year	Constants a	b	c
1 / 3	445	7	2,5
1 / 2	535	7	2,5
1	725	8	2,5
2	965	9	2,0
5	1 325	10	1,5
10	1700	11	0,5

Figure 2.1 *Intensity-Duration-Frequency curves for Lundby, Göteborg 1921-1939. After Arnell (1982).*

Table 2.1 *Data concerning selected rainfall events for design of detention basins, Lundby 1921-1939.*

	Average value	Standard deviation
Number per year	10	4
Volume per event (mm)	18.9	9.6
Duration per event (min)	582	426

The Average-Intensity-Duration (I-D-F) design storm is just the constant rainfall intensity obtained from the I-D-F curves (see Fig. 2.2). When a detention basin is designed, rainfalls with different durations are tested to find the duration that gives the maximum basin volume. 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. 2.3.

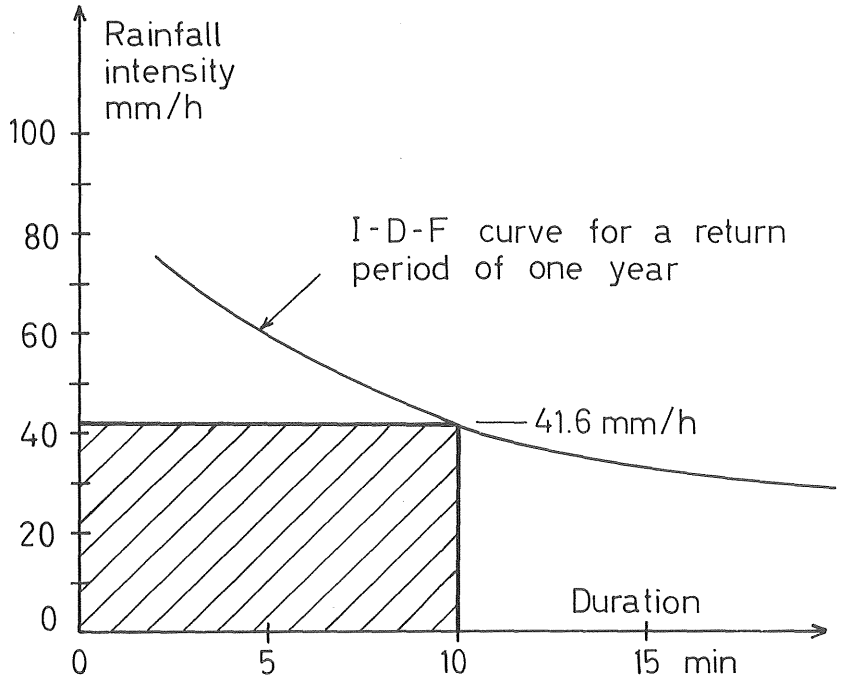


Figure 2.2 *Example of the evaluation of an Average-Intensity-Duration design storm from an Intensity-Duration-Frequency curve. A constant intensity of 41.5 mm/h during 10 minutes. Return period one year. After Arnell (1982).*

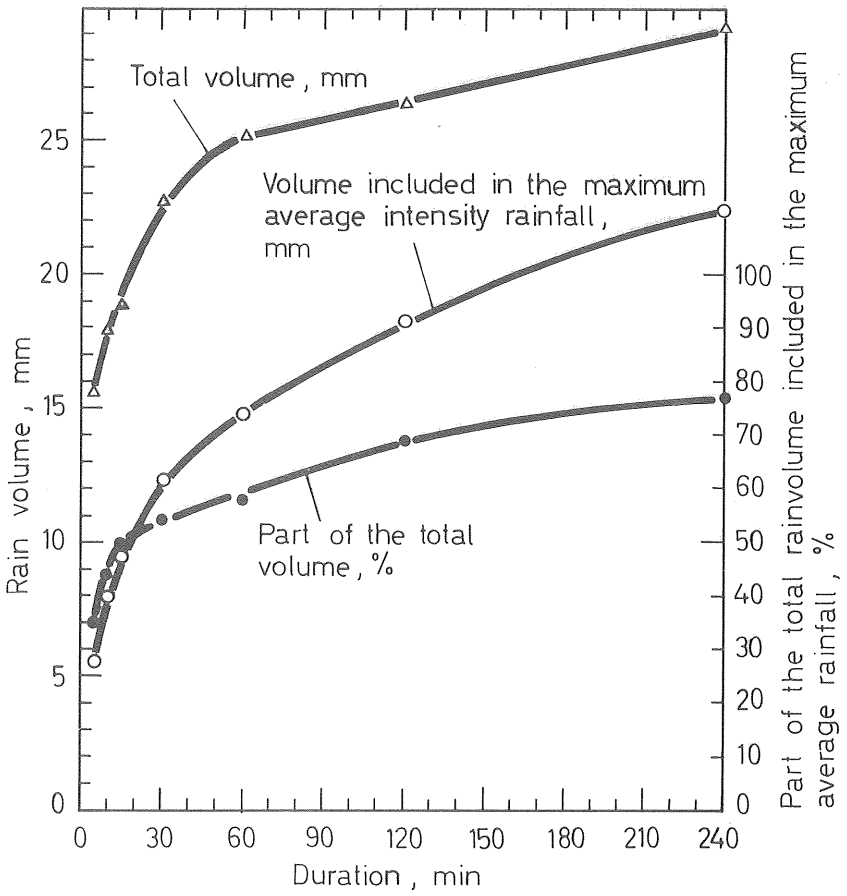


Figure 2.3 *The part of the total rain volume included in the maximum average intensity for different durations. Average values for rainfalls with a return period exceeding ½ year. Data from Lundby, Göteborg 1921-1939. After Arnell (1982).*

The Chicago design storm presented in 1957 by Keifer and Chu was developed from the mathematical expression for the I-D-F curves and its most important characteristic is that the maximum average intensities for all durations of the storm follow an I-D-F curve. The local Chicago storm used in the present investigation was derived from Eq. (2.3) and is described by the equations:

$$i = \frac{a \cdot b}{\left(\frac{|t-r\,T|}{r} + b\right)^2} + c \quad \text{(before the peak)} \qquad (2.4)$$

$$i = \frac{a \cdot b}{\left(\frac{|t-r\,T|}{1-r} + b\right)^2} + c \quad \text{(after the peak)} \qquad (2.5)$$

where

t = time

T = total duration chosen equal to 720 minutes

r = relationship between the time prior to peak intensity and the total duration T.

The value of r was evaluated from the local data for Lundby and is given in Table 2.2. An example of the local Chicago design storm is given in Fig. 2.4.

Table 2.2 *Average values of the relationship, r, between the time prior to peak intensity and the total duration of 720 minutes. Lundby, Göteborg, 1921-1939.*

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

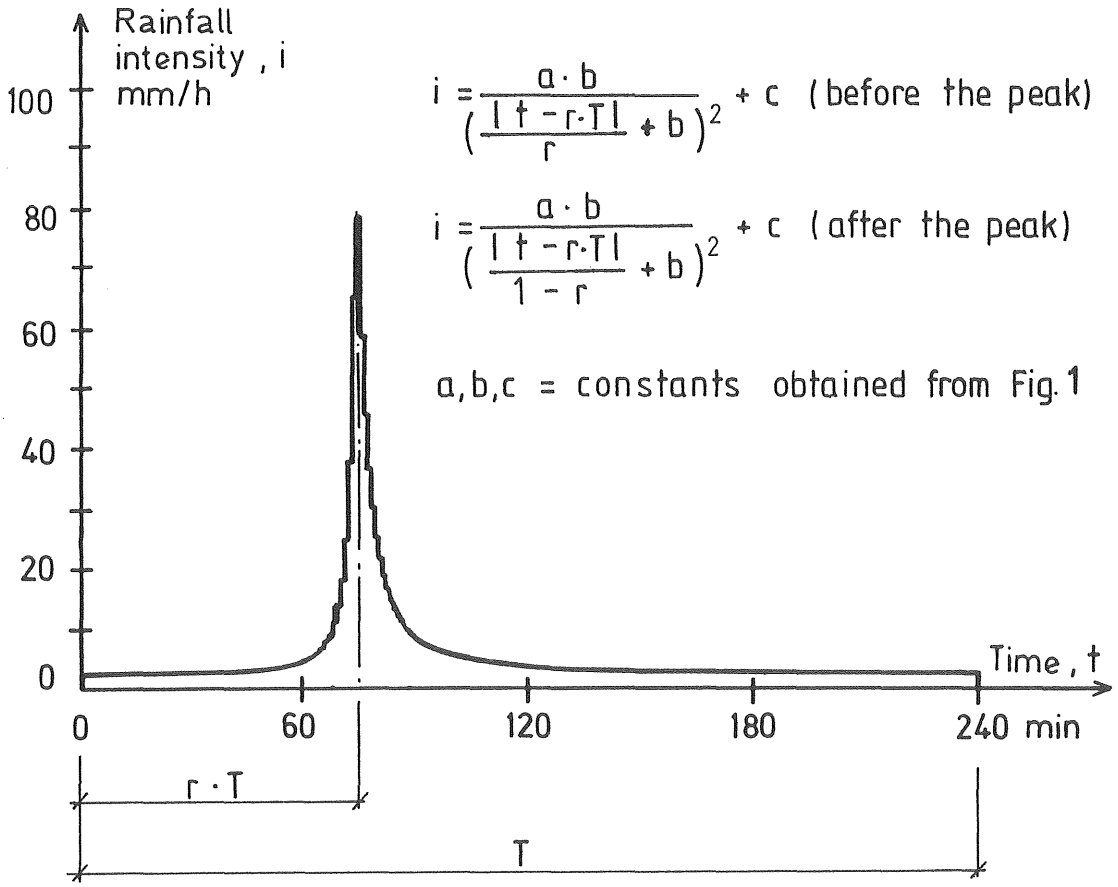


Figure 2.4 *Design rainfall, suggested by Keifer and Chu (1957), derived from the Intensity-Duration-Frequency relationship for Lundby, Göteborg, 1921-1939. Return period one year. After Arnell (1982).*

The Sifalda design storm (Sifalda, 1973) is compounded of the I-D-F design storm with precipitation added before and after the maximum duration (see Fig. 2.5). The advantage of the Sifalda

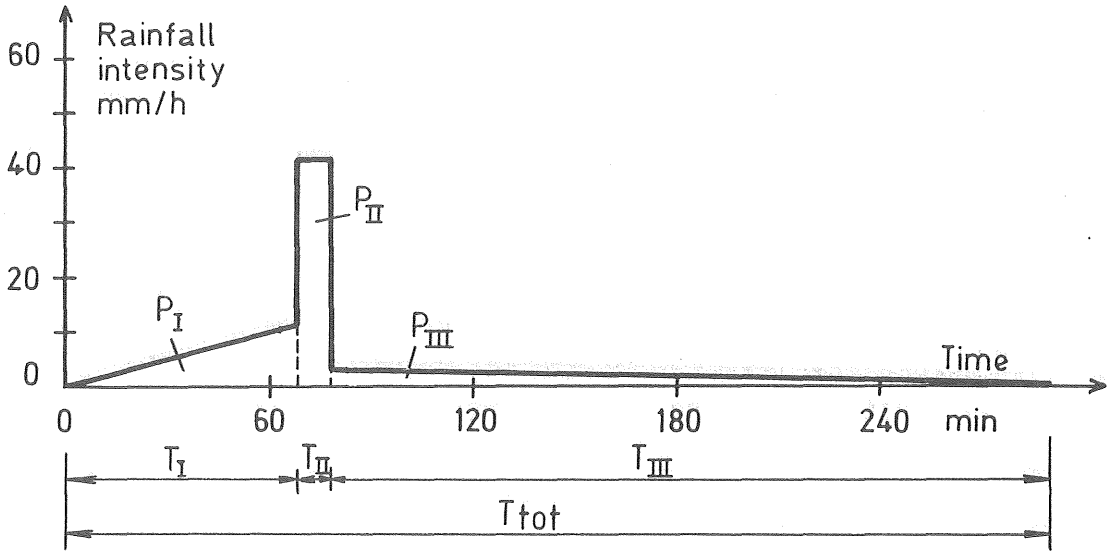


Figure 2.5 *Example of a design storm of the Sifalda-type, estimated from data for Lundby, Göteborg 1921-1939. Return period one year. After Arnell (1982).*

storm compared to the I-D-F design storm is that the total volume is more correct. For the local design storm for Lundby the total volume and duration and the volumes and durations of the parts before and after the main part were determined by regression analysis of the heaviest historical rainfalls for Lundby 1921-1939. Local design storms were estimated by the following equations (see also Fig. 2.6):

$$T_{tot} = u + v \cdot \ln T \qquad (2.6)$$

$$T_{III} = (e + f \cdot T)\, T_{tot}^{\frac{1}{100}} \qquad (2.7)$$

$$T_I = T_{tot} - T_{II} - T_{III} \qquad (2.8)$$

$$P_{tot} = g + h \cdot \ln T \qquad (2.9)$$

$$P_{III} = (k + l \cdot T)\, P_{tot}^{\frac{1}{100}} \qquad (2.10)$$

$$P_I = P_{tot} - P_{II} - P_{III} \qquad (2.11)$$

in which the definition of the symbols are given in Fig. 2.6 and the values of the constants are listed in Table 2.3. The distribution in time of the volumes was done with the added restriction that the intensities at the beginning and at the end of the rainfall are equal to 0.1 mm/h, which was the intensity value used for separation of the independent rainfalls.

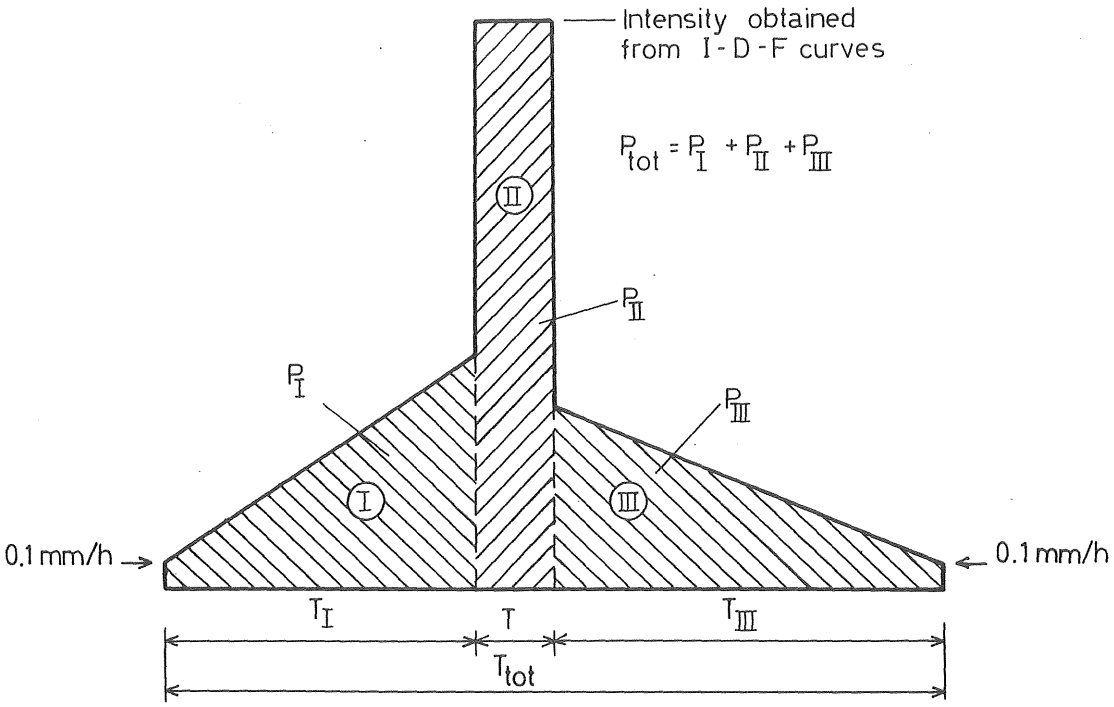


Figure 2.6 Definition of parameters of the local Sifalda design storm.

Further details concerning all the local design storms are given by Arnell (1982).

Table 2.3 *Constants for estimation of the local Sifalda design storm. Lundby, Göteborg, 1921-1939.*

Constants	Return periods (year)				
	1/3	1/2	1	2	5
u	291.67	238.07	125.25	111.17	74.34
v	46.77	55.67	73.09	62.49	70.85
e	56.73	59.72	76.40	75.31	67.70
f	-0.11	-0.14	-0.24	-0.24	-0.19
g	11.07	9.82	7.37	8.92	8.99
h	2.84	3.57	4.90	5.04	5.58
k	35.33	34.12	30.09	25.65	18.26
l	-0.12	-0.11	-0.09	-0.07	-0.03

2.3 Description of the Runoff Model Used

The inflow to the detention basins were calculated by the CTH-Urban Runoff Model (CTH-Model).

The CTH-Model is a typical design/analysis single-event model. The structure of the CTH-Model, which is shown in Fig. 2.7, 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.

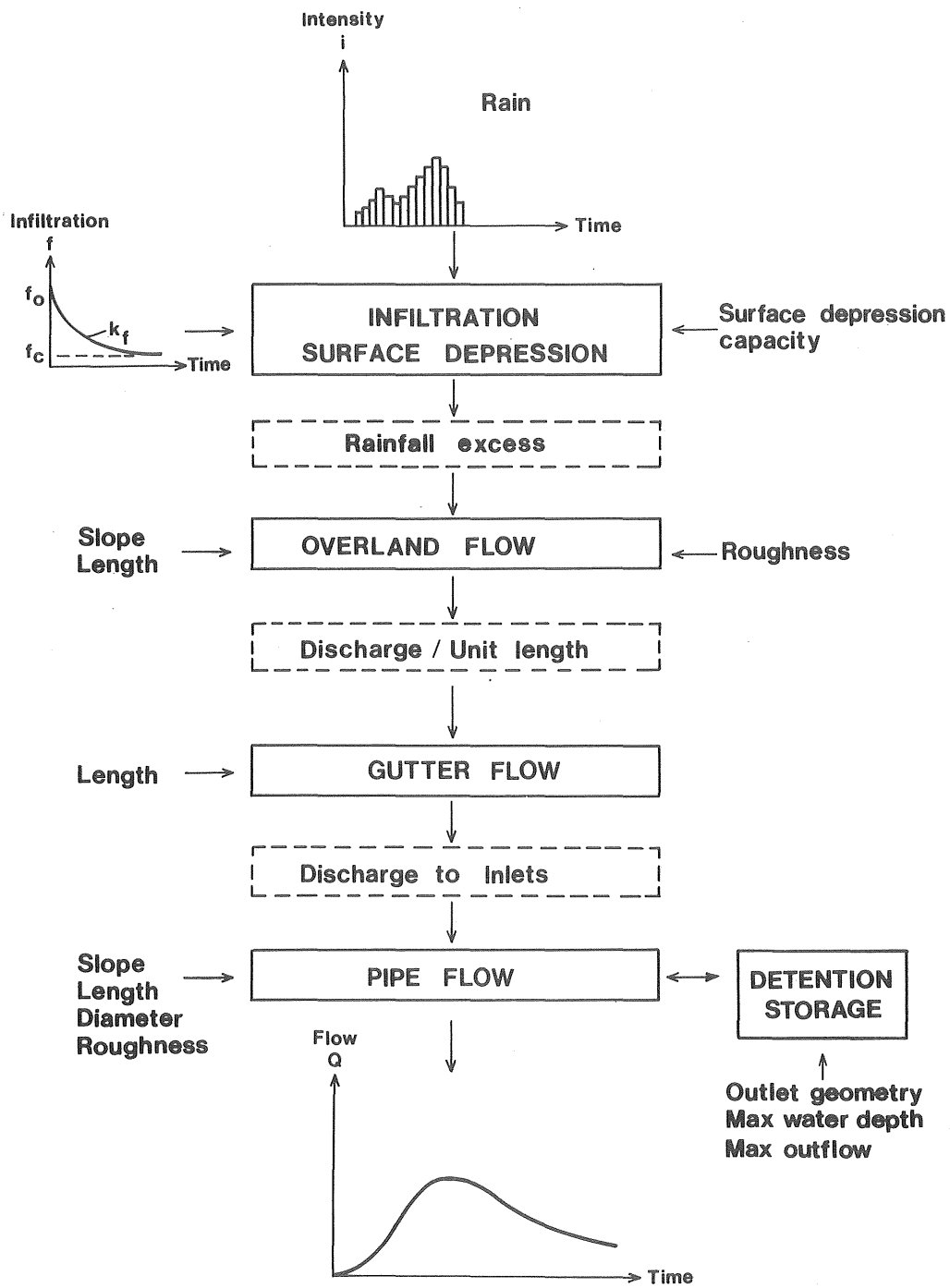


Figure 2.7 *Structure of the CTH-Urban Runoff Model. After Arnell (1980).*

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 not 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). Further information about the CTH-Model and its validation is reported by Arnell (1980).

The CTH-Model was used one time only on each rainfall in each test catchment. The inflow hydrographs were stored on a data file and later used for all different designs of the detention basins.

2.4 Description of the Detention Storage Model

The detention basins studied in this investigation are on-line basins where the water is flowing through the basins and the basins are emptied through outlet-holes of nozzle-type with atmospheric pressure at the down-stream sides (see Fig. 2.8). The outflows are functions of the water depths in the basins and the outflows reach their maximum values when the basins are full. Given baseflows are permitted to pass the basins before the real detention storage start. This is to maintain self-cleaning of the basins. The detention basins are assumed to cause no surcharging or backwater effects upstream of the basins.

The storages and the outflows were calculated with the equations:

$$\frac{dM}{dt} = Q_1 - Q_2 \qquad (2.12)$$

$$Q_2 = K\,H^m \qquad (2.13)$$

where

M	=	volume of water in the detention basin
t	=	time
Q_1 and Q_2	=	inflow to and outflow from the basin
H	=	water depth in the basin above the center of the outlet hole
K and m	=	constants which depend on the geometry of the outlet construction.

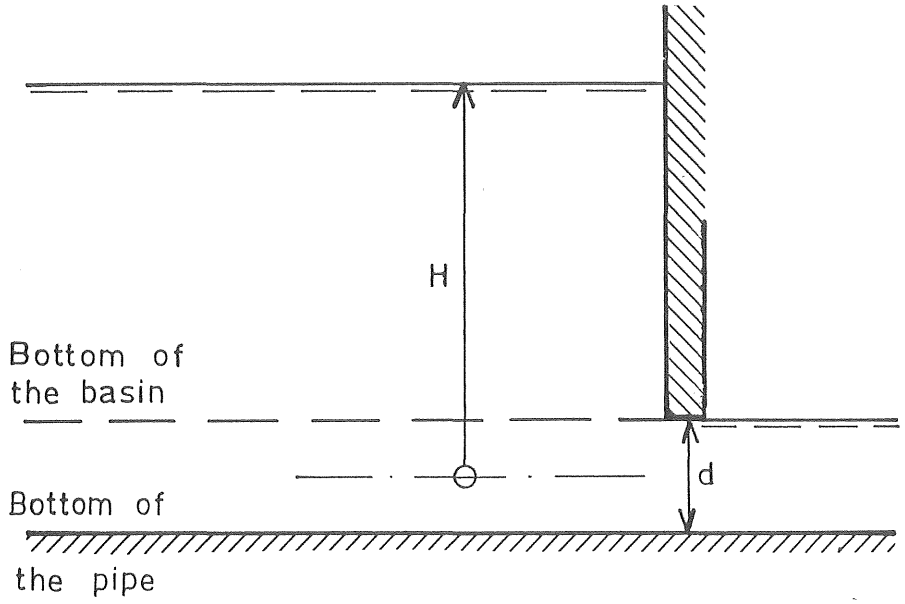


Figure 2.8 The outlet for a detention basin.

Eq. (2.12) is solved by the following finite difference scheme.

$$\frac{Q_1^j + Q_1^{j+1}}{2} - \frac{Q_2^j + Q_2^{j+1}}{2} = \frac{A_s(H^{j+1}-H^j)}{\Delta t} \qquad (2.14)$$

where

j and j+1 = time j and j+1

A_s = area of the water surface in the basin (constant, independent of water depth)

Δt = length of time step.

The outflow through the nozzle-type outlet hole is calculated by the equation

$$Q_2 = A_o \left(\frac{2g}{1+k_i}\right)^{\frac{1}{2}} \cdot H^{\frac{1}{2}} \qquad (2.15)$$

where

A_o = size of the outlet hole

g = acceleration of gravity

k_i = coefficient for calculation of head loss at the outlet hole.

The Eqs 2.14 and 2.15 are solved ny Newton-Raphson iterations.

The detention storage model includes a routine for design of the size of the horizontal area, A_s, of the basin given the maximum permitted water depth and outflow. The design is done in the following steps.

1. The size of the outlet hole, A_o, is calculated by Eq 2.15 when the maximum permitted water depth, the outflow, and the value of k_i are given (the value of k_i is obtained from hydraulic handbooks). Due to the risk for clogging the diameter of the hole should not be smaller than 150-200 mm.

2. In the present investigation the detention storage was selected to start when the water level reached the head of the outlet hole. The minimum flow for self-cleaning at that water level was estimated by insertion of $H = d/2$ (d = diameter of the outlet hole) in Eq 2.15.

3. The size of the horizontal area of the basin is determined by a sort of "trial and error routine". The area is determined by running the individual hydrographs through the basin and adjusting the area until the maximum water level reaches or comes close to the maximum permitted water depth. Usually 5-10 trials are necessary.

Data for the different basins are given in Section 2.5.

To check if there was any influence on the detention of water in the basin from one rainfall event to the next event the time necessary to empty the basin was calculated by the equation:

$$t_e = 2 \frac{A_s}{K} (H_e^{\frac{1}{2}} - H_{red}^{\frac{1}{2}}) \qquad (2.16)$$

where

t_e = time necessary to empty the basin from the water depth H_e at the end of the rainfall to the water depth H_{red} at the head of the outlet hole

$K = A_o(2g/(1+k_i))^{\frac{1}{2}}$.

2.5 Description of the Test Catchments and Detention Basins

As catchments for tests of the different types of rainfall data three areas were chosen, which had been used by Arnell (1980, 1982) in his validation of the CTH-Model, and in his study of rainfall data for design of sewer pipes. The areas have different sizes, topography, and types of buildings as shown in Table 2.4 and were chosen because input data was already prepared. Arnell (1980) has described each catchment in more detail.

Table 2.4 Test catchment data and summary of runoff-simulation input data.

Catchment data and runoff input data	Bergsjön	Linköping 1	Linköping 2
Size (km^2)	0.154	1.450	0.185
Impermeable part (%)	38	46	34
Land use	Apartment complexes	Mixed housing and commercial buildings	Single family detached houses
Slopes	Steep	Flat	Medium
Number of pipes used in the simulations	73	125	54
Number of inlets used in the simulations	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
Length of the main pipe (m)	606	2824	845
Average slope of the main pipe	0.045	0.0063	0.012

The input data used for the runoff simulations in the three catchments, and listed in Table 2.4, were the same as those used by Arnell (1980). 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.

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 not too high rainfall intensities. Lyngfelt (1981) has tested if the runoff volumes for the 40 most intense rainfalls during two years in the test catchments 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. Underestimated simulated flow values for these rainfalls can be expected in the present study. This will affect the statistical distribution functions of the basin volumes 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 of 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. 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.

One detention basin is located at the outlet of each catchment. The maximum permitted water depth of each basin was varied between 2.0 m and 3.5 m above the center of the outlet hole which combined with the maximum permitted outflows of 5 to 30 $l/s \cdot ha_{red}$ gave the detention basin data listed in Tables 2.5 and 2.6. The outflow given in $l/s \cdot ha_{red}$ is counted for the contributing areas only. The value of the head loss (k_i) at the outlet was chosen equal to 0.5.

The diameters of the outlet holes given in Tables 2.5 and 2.6 are in some cases smaller than is recommended, due to the risk for clogging (see Section 2.4), but the listed diameters are used despite making the results comparable.

The detention of water starts when the inflow is larger than the minimum flow for self-cleaning which appears when the water reaches the head of the outlet hole. Thus, the available storage depth is 2.0 m and 3.5 m respectively minus half of the diameter of the outlet hole.

Table 2.5 Detention basin data. Maximum permitted water depth 2.0 m.

Basin data	Bergsjön	Linköping 1	Linköping 2
Maximum permitted water depth (m)	2.0	2.0	2.0
Maximum permitted outflow = 5.0 $l/s \cdot ha_{red}$			
Maximum outflow (m^3/s)	0.0201	0.2465	0.0286
Diameter of the outlet hole (m)	0.071	0.248	0.084
Minimum flow for self-cleaning (m^3/s)	0.0027	0.0613	0.0042
Maximum permitted outflow = 10 $l/s \cdot ha_{red}$			
Maximum outflow (m^3/s)	0.0401	0.4930	0.0571
Diameter of the outlet hole (m)	0.100	0.350	0.119
Minimum flow for self-cleaning (m^3/s)	0.0063	0.1459	0.0099
Maximum permitted outflow = 20 $\cdot l/s\ ha_{red}$			
Maximum outflow (m^3/s)	0.0802	0.9860	0.1142
Diameter of the outlet hole (m)	0.141	0.495	0.169
Minimum flow for self-cleaning (m^3/s)	0.0151	0.3470	0.0234
Maximum permitted outflow = 30 $\cdot l/s\ ha_{red}$			
Maximum outflow (m^3/s)	0.1203	1.4790	0.1713
Diameter of the outlet hole (m)	0.173	0.607	0.207
Minimum flow for self-cleaning (m^3/s)	0.0250	0.5761	0.0389

Table 2.6 *Detention basin data. Maximum permitted water depth 3.5 m.*

Basin data	Bergsjön	Linköping 1	Linköping 2
Maximum permitted water depth (m)	3.5	3.5	3.5
Maximum permitted outflow = 5.0 $l/s \cdot ha_{red}$			
Maximum outflow (m^3/s)	0.0201	0.2465	0.0286
Diameter of the outlet hole (m)	0.062	0.215	0.073
Minimum flow for self-cleaning (m^3/s)	0.0019	0.0432	0.0029
Maximum permitted outflow = 10 $l/s \cdot ha_{red}$			
Maximum outflow (m^3/s)	0.0401	0.4930	0.0571
Diameter of the outlet hole (m)	0.087	0.305	0.104
Minimum flow for self-cleaning (m^3/s)	0.0045	0.1028	0.0069
Maximum permitted outflow = 20 $l/s \cdot ha_{red}$			
Maximum outflow (m^3/s)	0.0802	0.9860	0.1142
Diameter of the outlet hole (m)	0.123	0.431	0.147
Minimum flow for self-cleaning (m^3/s)	0.0106	0.2446	0.0165
Maximum permitted outflow = 30 $l/s \cdot ha_{red}$			
Maximum outflow (m^3/s)	0.1203	1.4790	0.1713
Diameter of the outlet hole (m)	0.150	0.528	0.180
Minimum flow for self-cleaning (m^3/s)	0.0176	0.4060	0.0274

2.6 Design of the Detention Basins for Various Types of Rainfall Data

Design Using Historical Storms

In each test catchment the basins were designed for the 100 largest historical storms for each outflow level found through the screening of the total rainfall series described in Section 2.2. Since many of the storms were the same for the different outflow levels the total number of storms were reduced to 176. These 176 storms were first run through the CTH-Model and the inflows to the basins for all storms were stored on a computer file. That file was then used for the design of the basins for different outflow levels. Thus, the runoff for the different storms for each basin needed to be calculated only once.

When doing the design it was controled that the basin was emptied after the preceding rainfall.

Through the design the necessary volume of the basin for each storm was determined to prevent the basin from flooding. A series of basin volumes for each outflow level was obtained corresponding to the 176 rainfalls. These basin volumes were ranked in descending order and the 36 largest (corresponding to a return period of six months) were plotted on a statistical probability paper using the plotting formula

$$y_i = \sum_{j=1}^{i} \frac{1}{N + 1 - j} \quad ; \quad i = 1, 2, \ldots\ldots, N \qquad (2.17)$$

where

y_i = plotting position for the basin volumes in increasing order (y_i = lnF; where F is the return period)

N = number of treated basin volumes (in this case 36).

The results are shown in Appendix I.

Design Using Design Storms

The different basin volumes have also been determined for the following design storms:

(1) Average-Intensity-Duration design storm
(2) Chicago design storm
(3) Sifalda design storm.

The designs were carried out for the return periods of six months, 1, 2, and 5 years. For each return period designs were made for different durations of the I-D-F design storm and of part II of the Sifalda design storm to find the duration that gave the maximum basin volume.

For each design storm and each return period one maximum basin volume was obtained for each outflow level. The result is plotted on the graphs given in Appendix I.

In Table 2.7 are listed the durations that caused the maximum basin volumes and for a return period of one year for the different design storms and outflow levels.

Table 2.7 *The storm durations (for the Sifalda storm duration of part II) that caused the maximum basin volumes for the I-D-F design storm, the Sifalda design storm, and for traditional design. Return period one year. Values in minutes.*

Catchment	Maximum permitted outflow $l/s \cdot ha_{red}$	Design storm and maximum permitted water depth (m) I-D-F 2.0	I-D-F 3.5	Sifalda 2.0	Sifalda 3.5	Traditional des. 2.0	Traditional des. 3.5
Bergsjön	5.0	360	360	240	240	360	360
	10.0	110	120	110	110	120	200
	20.0	35	35	40	35	70	70
	30.0	25	25	25	25	30	35
Linköping 1	5.0	360	360	240	240	360	360
	10.0	120	120	110	140	120	120
	20.0	60	60	50	40	50	60
	30.0	35	35	30	30	25	35
Linköping 2	5.0	360	360	240	240	360	360
	10.0	120	120	140	140	120	200
	20.0	35	35	35	45	70	70
	30.0	30	25	25	25	30	35

2.7 Traditional Design of the Detention Basins

The Swedish Water and Waste Water Works Association has published a report describing the design of detention basins (Bergström, 1976). Stahre (1981) has further developed the method, which includes a linear time-area diagram for the estimation of the inflow to the basin, rainfall data obtained from the I-D-F relationships, and a constant outflow from the basin. The general equation for calculation of the volume of the basin is

$$M = 0.06 \left(i_m \cdot T - Q_2^c \cdot T - Q_2^c \cdot t_c + \frac{(Q_2^c)^2 \, t_c}{i_m}\right) \qquad (2.18)$$

where

M = volume of the basin (m^3/ha_{red})
t_c = time of concentration (min)
i_m = rainfall intensity obtained from Fig. 2.1 or Eq. (2.3) ($l/s \cdot ha_{red}$)
T = duration of the rainfall (min)
Q_2^c = constant outflow ($l/s \cdot ha_{red}$)

The maximum volume of the basin is obtained when

$$\frac{dM}{dT} = 0 \qquad (2.19)$$

The outflows were in this investigation approximated by the equation

$$Q_2^c = \frac{Q_{max} + Q_{min}}{2} \qquad (2.20)$$

where

Q_{max} = maximum outflow when the basin is full
Q_{min} = flow for which the storage starts.

The values of Q_{max} and Q_{min} were taken from Tables 2.5 and 2.6 and the resulting values of the outflows Q_2^c are listed in Table 2.8.

Table 2.8 *Outflows for use in design with the traditional method.* $Q_2^c = (Q_{max} + Q_{min})/2$. *Values in l/s* ha_{red}.

Catchment	Maximum permitted outflow ($l/s \cdot ha_{red}$)			
	5.0	10.0	20.0	30.0
Water depth 3.5 m				
Bergsjön	2.74	5.56	11.32	17.21
Linköping 1	2.94	6.04	12.48	19.12
Linköping 2	2.77	5.60	11.45	17.41
Water depth 2.0 m				
Bergsjön	2.84	5.79	11.88	18.12
Linköping 1	3.12	6.48	13.52	20.84
Linköping 2	2.87	5.87	12.05	18.41

The time of concentration, t_c, was calculated by Eq. (2.21) given by Lyngfelt (1981)

$$t_c = k1 \frac{L_{h80}^{pL}}{i_m^{pi} \cdot A_{red}^{pA} \cdot S_h^{pS}} \qquad (2.21)$$

where

L_{h80} = length of the main pipe plus 80 m (m)
A_{red} = total sizes of contributing areas (ha)
S_h = average slope of the main pipe.

Values of L_{h80}, A_{red}, and S_h were taken from Table 2.4, and values of the constants k1, pL, pi, pA and pS were taken from Table 2.9.

The basin volumes were calculated by changing the rainfall duration stepwise to find the one that gave the maximum volumes. The results are listed in Tables 3.5 and 3.6 and plotted in Appendix I for the two maximum water depths of 2.0 m and 3.5 m for each basin.

Table 2.9 *Values of constants for estimation of time of concentration by Eq. (2.21). After Lyngfelt (1981).*

Constants	Catchment	
	Bergsjön Linköping 2	Linköping 1
k_1	0.490	0.079
pL	0.50	0.71
pi	0.32	0.32
pA	0.10	0.05
pS	0.26	0.35

3 COMPARISON OF DESIGN FOR VARIOUS TYPES OF RAINFALL DATA

3.1 Factors Influencing the Comparison of Design for Different Types of Rainfall Data

The following factors influence the interpretation of the result.

* The errors and uncertainties inherent in the modeling of urban runoff due to the models capability of simulating the real runoff process.

* The errors and uncertainties obtained through the selection of input data describing the runoff basin.

* The errors and uncertainites in the rainfall data.

* The changes in return periods of the basin volumes due to over- and underestimations of the volumes.

The CTH-Model has been shown (Arnell, 1980) to simulate the runoff volumes and the peak flows within a model error of about ±15% after calibration of the sizes of contributing areas. In the present study the same test basins were utilized as were used for validation of the model, so the same accuracy can be expected. This also includes the uncertainties obtained through the selection of model input data.

The different design storms are related to the Intensity-Duration-Frequency relationships. Arnell (1982) has estimated the uncertainties of the I-D-F curves shown in Fig. 2.1 to be ±7-14%. He also shows that changes in the rainfall intensities caused approximately the same changes in the calculated peak flows. Calculated runoff volumes must be influenced in the same way.

The interpretation of the resulting basin volumes for the different types of rainfall data should also be based on the costs for over- and underestimation of the basin volumes. The cost includes investment costs and flooding costs. Stahre (1981) has published a few figures on investment costs shown in Fig. 3.1, but the values are too vague to make possible an estimation of the increase

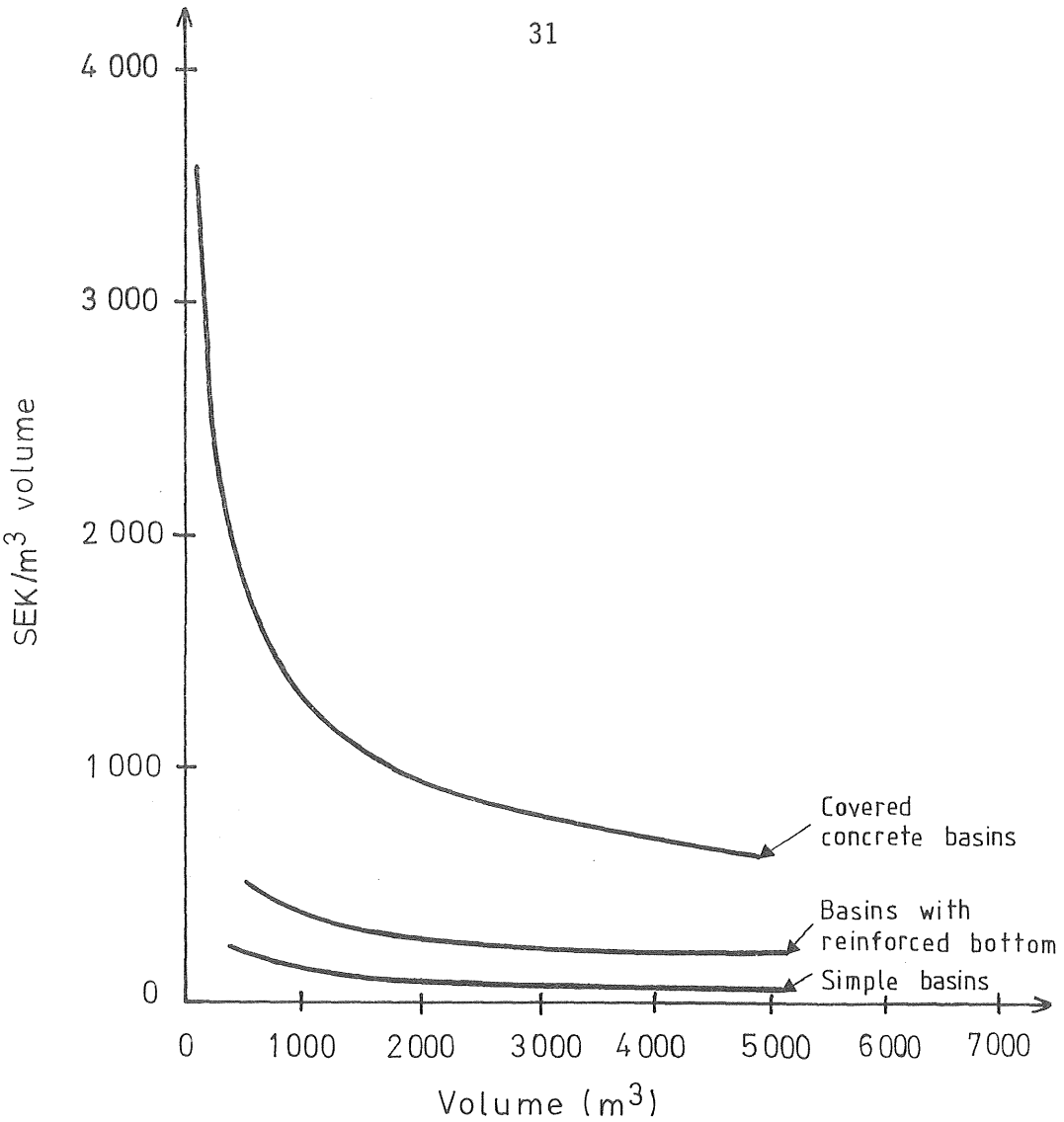


Figure 3.1 Investment costs for different types of detention basins (after Stahre, 1981).

of investment costs due to an increase of the basin volumes. However, the figure shows that for large basins the investment costs per m^3 are directly proportional to the size of the basins, while for smaller basins the costs per m^3 increase when the volumes decrease.

The changes in return periods of the basin volumes due to over- and underestimation of the basin volumes can be found in the figures shown in Appendix I. It can also be estimated if the statistical distribution functions shown in Appendix I are expressed by the equation,

$$\ln M = m + z \cdot \ln F \qquad (3.1)$$

where

M = basin volume

F = return period

m = lower boundary value for the distribution function

z = slope of the distribution function

Eq. (3.1) is approximately valid for return periods less than 5-10 years. For two different return periods the equation can be changed to

$$F_2 = \left(\frac{M_2}{M_1}\right)^{1/z} \cdot F_1 \qquad (3.2)$$

where

F_1, F_2 = return periods corresponding to basin volumes M_1 and M_2.

The value of z is approximately 0.31-0.34 for maximum outflows of 5-10 $l/s \cdot ha_{red}$ and 0.40-0.50 for outflows of 20-30 $l/s \cdot ha$. Thus, an over- or underestimation of the basin volume of 10% changes the return period by approximately 20-30%.

The different uncertainties and errors are of the same size as were estimated by Arnell (1982) in a similar study for estimation of peak flow values and, thus, the same conclusions can be drawn.

* The systematic differences between the basin volumes for design storms and the basin volumes for historical storms should be as small as possible.

* The standard deviations of the differences between the basin volumes 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.

Besides these factors the physical and statistical characteristics of the design storms and their development bust be included in the interpretation.

3.2 Result of Design Using Historical Storms

The complete results of design using historical storms are given in Appendix I. The basin volumes corresponding to the return periods of six months, 1, 2, and 5 years are listed in Table 3.1. The volumes in Table 3.1 were estimated by linear interpolation between the plotted points in Appendix I.

The basin volumes obtained for the historical storms are assumed to be the most correct. No theoretical distributions were fitted to the data becasue the plotted points themselves are the most correct values available.

Table 3.1 Basin volumes calculated for the historical storms (m^3/ha_{red}).

Catchment	Maximum permitted outflow $l/s \cdot ha_{red}$	Maximum basin depth 2.0 m Return period ½	1	2	5	3.5 m Return period ½	1	2	5
Bergsjön	5	145	195	248	318	144	188	242	310
	10	116	136	175	242	112	135	173	238
	20	68	103	129	164	69	103	123	164
	30	53	82	102	150	52	83	104	149
Linköping 1	5	141	188	215	323	145	191	250	323
	10	105	125	166	230	106	127	170	235
	20	56	85	110	147	59	90	115	156
	30	31	63	80	122	34	65	85	127
Linköping 2	5	140	187	241	308	139	186	239	310
	10	110	129	164	231	109	131	168	232
	20	61	97	120	157	62	97	119	160
	30	45	79	97	140	45	78	98	140

The basin volumes obtained for the historical storms and listed in Table 3.1 show that very small differences in basin volumes were obtained for the two basin depths 2.0 m and 3.5 m, indicating that the variation in basin volumes with maximum permitted water depth is small.

For the smallest outflow 5 l/s·ha approximately the same basin volumes were obtained for the three catchments. For the outflow of 30 l/s·ha, differences of about 20% between the basin sizes for the small Bergsjön basin and the larger Linköping 1 basin were found. The explanation is that for small outflows the rain volumes are the most important factor, while for larger outflows both rain volumes and variation in the inflow hydrographs to the detention storages are important. Thus, it is important to use a runoff model to calculate the inflows to the detention basins for larger catchments and outflows, which conclusion was also drawn by Johansen (1979).

The screening of the rainfall series (see Section 2.2) used to reduce the number of rainfall events to be used in the runoff simulations gave a sufficient number of rainfalls to make possible a correct statistical design of the basin volumes for return periods of six months and longer.

The 176 rainfalls used in the present study include all the 110 rainfalls used by Arnell (1982) in a similar study for estimation of peak-flows and design of pipes. The conclusion is that the 176 rainfalls can be used both for design of sewer pipes and detention basins for return periods of six months and longer.

3.3 Result of Design Using Design Storms

The basin volumes for the design storms and for the different design return periods were plotted in the same diagrams in Appendix I as where the basin volumes for the historical storms were plotted. Over- and underestimations of basin volumes obtained for the design storms were estimated by comparing with the basin volumes obtained for the historical storms. The resulting differences are listed in Tables 3.2 and 3.3. Table 3.4 gives the sum-

mary of the result as mean values and standard deviations of the differences for the different design storms, runoff areas and maximum water depths.

Table 3.2 *Deviations in percent between the basin volumes for design storms and the basin volumes for historical storms. MV is the mean value and σ the standard deviation. Maximum basin depth 2.0 m.*

Catchment	Maximum permitted outflow l/s ha_{red}	I-D-F Design Storm Return period ½	1	2	5	Chicago Design Storm Return period ½	1	2	5	Sifalda Design Storm Return period ½	1	2	5	Traditional design I-D-F curves and time-area method Return period ½	1	2	5
	5	-16	-19	-24	-20	-11	-16	-23	-21	+32	+23	+11	-3	-10	-12	-17	-14
	10	-27	-17	-18	-23	-20	-10	-13	-14	+4	+21	+13	-4	-17	-10	-13	-17
Bergsjön	20	-18	-22	-17	-10	-4	-14	-9	-1	+6	+10	+12	+9	-7	-15	-12	-7
	30	-17	-21	-11	-13	-4	-12	-5	-7	-6	+5	+15	+1	-8	-12	-4	-8
	MV;σ	-18;	4.6			-12;	6.4			+9;	10.3			-11;	4.0		
	5	-15	-16	-23	-21	-10	-13	-22	-22	+37	+29	+15	-4	-14	-15	-22	-21
	10	-25	-14	-17	-22	-16	-6	-11	-13	+12	+29	+17	0	-22	-14	-17	-21
Linköping 1	20	-23	-22	-16	-12	-2	-6	-3	+3	+9	+19	+20	+13	-21	-20	-15	-8
	30	-19	-27	-13	-11	+23	-5	+8	+5	+19	+11	+25	+9	-16	-24	-9	-7
	MV;σ	-19;	4.9			-6;	11.6			+16;	10.6			-17;	5.3		
	5	-14	-17	-23	-19	-10	-14	-22	-20	+35	+27	+13	-2	-9	-9	-15	-12
	10	-25	-15	-15	-21	-18	-7	-9	-12	+7	+24	+18	-2	-15	-7	-9	-15
Linköping 2	20	-15	-22	-14	-10	+3	-11	-6	+1	+11	+12	+16	+10	-2	-12	-8	-4
	30	-13	-23	-12	-11	+9	-13	-3	-4	+4	+4	+15	+4	+2	-14	-3	-4
	MV;σ	-17;	4.7			-9;	8.4			+12;	10.2			-9;	5.2		
		MV = -17.9; σ = 4.7				MV = -8.5; σ = 9.2				MV = +12.6;σ = 10.6				MV = -12.2; σ = 5.9			

Table 3.3 *Deviations in percent between the basin volumes for design storms and the basin volumes for historical storms. MV is the mean value and σ the standard deviation. Maximum basin depth 3.5 m.*

Catchment	Maximum permitted outflow l/s ha_{red}	I-D-F Design Storm Return period ½	1	2	5	Chicago Design Storm Return period ½	1	2	5	Sifalda Design Storm Return period ½	1	2	5	Traditional design I-D-F curves and time-area method Return period ½	1	2	5
	5	-15	-16	-22	-18	-9	-12	-20	-19	+35	+29	+14	±0	-9	-9	-14	-12
	10	-23	-16	-17	-21	-16	-9	-10	-13	+9	+21	+14	-2	-13	-8	-10	-15
Bergsjön	20	-17	-21	-12	-10	-3	-12	-3	±0	+6	+11	+19	+10	-6	-13	-6	-5
	30	-15	-20	-13	-11	±0	-11	-4	-5	±0	+6	+14	+3	-2	-11	-4	-6
	MV;σ	-17;	4.0			-9;	6.2			+12;	10.4			-9;	3.9		
	5	-17	-16	-24	-21	-10	-13	-22	-21	+35	+29	+12	-3	-15	-16	-21	-19
	10	-24	-15	-18	-22	-14	-5	-11	-12	+14	+29	+16	-1	-19	-12	-16	-20
Linköping 1	20	-25	-24	-18	-15	-2	-8	-3	±0	+10	+16	+17	+9	-19	-20	-15	-12
	30	-24	-26	-15	-12	+18	-3	+5	+2	+15	+14	+24	+9	-12	-20	-8	-7
	MV;σ	-20;	4.5			-6;	10.0			+15;	10.2			-16;	4.4		
	5	-14	-17	-23	-20	-8	-13	-21	-21	+36	+27	+14	-3	-6	-9	-14	-12
	10	-24	-15	-15	-21	-16	-8	-10	-12	+9	+23	+15	-2	-12	-6	-8	-13
Linköping 2	20	-15	-21	-13	-11	+3	-9	-3	+6	+13	+13	+18	+9	±0	-10	-5	-4
	30	-11	-22	-12	-10	+11	-9	-2	-2	+7	+8	+16	+6	+4	-9	-1	-2
	MV;σ	-17;	4.7			-7;	9.0			+13;	10.0			-7;	5.1		
		MV = -17.6; σ = 4.6				MV = -7.5; σ = 8.5				MV = +13.4;σ = 10.1				MV = -10.4; σ = 5.9			

Table 3.4 *Deviations in percent between the basin volumes for design storms and the basin volumes for historical storms. The mean values (MV) and standard deviations (σ) are for an average of all return periods and all maximum outflows.*

Rainfall data	Maximum water depth (m)	Bergsjön		Linköping 1		Linköping 2		All basins	
		MV	σ	MV	σ	MV	σ	MV	σ
I-D-F	2.0	-18	4.6	-19	4.9	-17	4.7	-18	4.7
design storm	3.5	-17	4.0	-20	4.5	-17	4.7	-18	4.6
Chicago	2.0	-12	6.4	- 6	11.6	- 9	8.4	- 9	9.2
design storm	3.5	- 9	6.2	- 6	10.0	- 7	9.0	- 8	8.5
Sifalda	2.0	+ 9	10.3	+16	10.6	+12	10.2	+13	10.6
design storm	3.5	+12	10.4	+15	10.2	+13	10.0	+13	10.1
Traditional design									
I-D-F curves and	2.0	-11	4.0	-17	5.3	- 9	5.2	-12	5.9
time-area method	3.5	- 9	3.9	-16	4.4	- 7	5.1	-10	5.9

The basin volumes corresponding to the I-D-F design storm are on average 18% smaller than the volumes obtained for the historical storms. The underestimations are slightly smaller for the larger outflows than for the smaller outflows. The too small basin volumes for the I-D-F design storm are mainly due to the total rainfall volumes for the I-D-F design storms which are smaller than the total volumes of the historical storms used for estimation of the I-D-F curves. When I-D-F curves are estimated the maximum intensity parts only of the real storms are regarded. The rainfall coming prior to and after the maximum parts are ignored and thus giving the I-D-F design storm a too small total volume, see Fig. 2.3 and Arnell (1982).

The standard deviations of the differences for the I-D-F design storms are small which means that there is a small variation in the result only and that the conclusion is clear: The I-D-F design storm gives an underestimation of the basin volumes.

The use of the Chicago design storm resulted in underestimated basin volumes of on average 9%. For the outflow of 5 l/s.ha the underestimations were 15-20% and for the outflow of 30 l/s.ha the volumes were both over- and underestimated. The explanation for the underestimations is mainly the same as for the I-D-F design storm, namely the total volume of the Chicago design storm is smaller than the total volumes of the historical storms used for evaluation of the design storm. However, it is slightly better than the I-D-F design storm because of the long total duration of the storm.

The use of the Chicago design storm results in more correct basin volumes for larger outflows which is logical because the high peak intensity of the storm has a greater effect on the basin volumes for large outflows than for small outflows.

When using the Sifalda design storm overestimated detention basin volumes of on average 13% were obtained. The overestimations were larger for the Linköping 1 basin than for the smaller basins. It was also larger for the small outflow of 5 l/s.ha than for the outflow of 30 l/s.ha.

The overestimated basin volumes for the Sifalda design storm were obtained mainly because the volumes of part I of the storms are too large. The explanation is found in the estimation of the regression equations (2.6-2.11) describing the storm, see Arnell (1982). It was assumed important that the total volumes of the design storm are close to the volumes of the historical storms which is expressed by Eq (2.9). For the volumes of part I and III a regression equation for part III was chosen because a higher value than for part I of the linear correlation coefficient between the volumes of part III and the durations of part II was obtained. Since the total volume and the volume of part III are given, and the volume of part II taken from the I-D-F curves is smaller than the average value of the historical storms, the volume of part I will be too large. This will result in an overestimation of the detention storages.

An improvement of the Sifalda design storm could be achieved if the regression could be based on part I instead of part III or if both part I and part III were used.

The overestimations for the Sifalda storm are less for the return period 5 years than for the other return periods tested, probably because the volume of part II of the storm is larger compared to the volumes of part I and III for the return period 5 years than for the shorter return periods.

3.4 Result of Design by the Traditional Method

The basin volumes calculated by the traditional method and described in Section 2.7 are listed in Tables 3.5 and 3.6. The volumes are also plotted in Appendix I together with the resulting basin volumes from design with historical storms and design storms. Over- and underestimations of the basin volumes obtained by the traditional method compared with the volumes obtained by the historical storm are listed in Tables 3.2 and 3.3, and a summary of the results are given in Table 3.4.

As can be seen in Tables 3.5 and 3.6 negligible differences in basin volumes were obtained for the two water depths 2.0 m and 3.5 m.

Table 3.5 *Volumes of the detention basins obtained with the traditional method expressed by Eq. (2.18). Maximum water depth 2.0 m. The volumes are given in m^3/ha_{red}.*

Return period years	Catchment	Maximum permitted outflow ($l/s \cdot ha_{red}$)			
		5.0	10.0	20.0	30.0
F=½	Bergsjön	130	96	63	49
	Linköping 1	121	82	44	26
	Linköping 2	128	94	60	46
F=1	Bergsjön	171	122	88	72
	Linköping 1	160	108	68	48
	Linköping 2	170	120	85	68
F=2	Bergsjön	206	152	114	98
	Linköping 1	191	138	94	73
	Linköping 2	204	150	111	94
F=5	Bergsjön	272	200	153	138
	Linköping 1	255	182	135	114
	Linköping 2	270	197	150	135

Table 3.6 *Volumes of the detention basins obtained with the traditional method expressed by Eq. (2.18). Maximum water depth 3.5 m. The volumes are given in m^3/ha_{red}.*

Return period years	Catchment	Maximum permitted outflow (l/s ha_{red})			
		5.0	10.0	20.0	30.0
F=½	Bergsjön	131	98	65	51
	Linköping 1	123	86	48	30
	Linköping 2	130	96	62	47
F=1	Bergsjön	171	124	90	74
	Linköping 1	161	112	72	52
	Linköping 2	169	123	87	71
F=2	Bergsjön	208	155	116	100
	Linköping 1	197	142	98	78
	Linköping 2	206	154	113	97
F=5	Bergsjön	274	203	156	140
	Linköping 1	262	188	138	118
	Linköping 2	272	201	153	137

The basin volumes were underestimated by, on average, 10-12%. The underestimations are larger for the larger Linköping 1 catchment than for the smaller catchments probably because the routing of the hydrographs are more important in a large catchment than in a small one. The underestimations are also larger for the small outflows of 5 $l/s \cdot ha_{red}$ and 10 $l/s \cdot ha_{red}$ than for the other outflows.

Sjöberg and Mårtensson (1982) obtained underestimations of approximately 15-40% when comparing design of percolation basins by historical storms with design by the traditional method. It is clear that the traditional method gives underestimations that are larger for small outflows than for larger outflows. A summary of the results obtained in the present study and the results obtained by Mårtensson and Sjöberg are shown in Fig. 3.2.

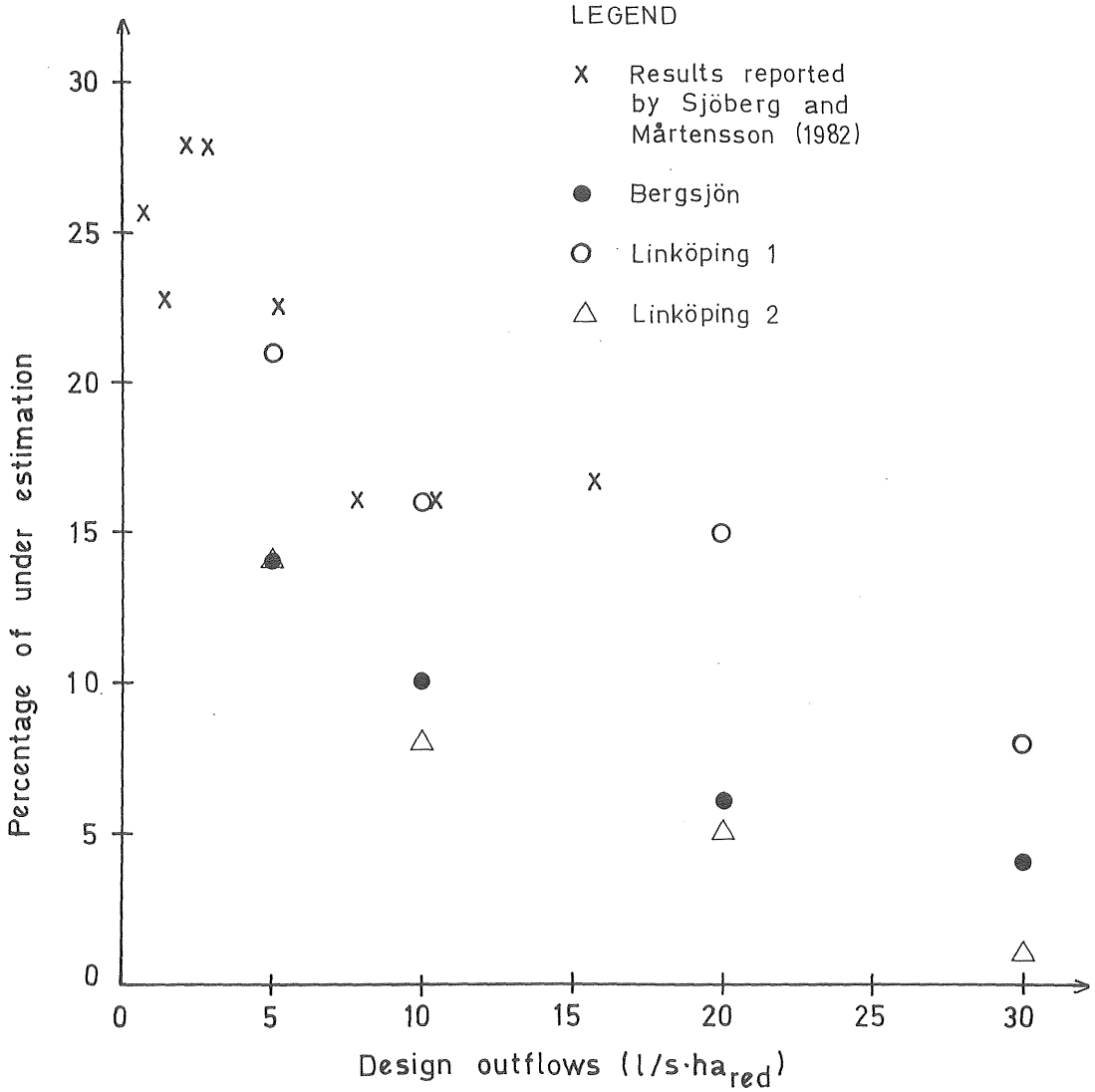


Figure 3.2 *Size of underestimations of detention basin volumes obtained by design with the traditional method. Results of the present study and results reported by Sjöberg and Mårtensson (1982). Return period 2 years.*

To compensate for an expected underestimation the basin volumes obtained by the traditional method must be increased by 5-20% depending on the permitted maximum outflow.

The durations of the rainfall for which the maximum volumes were obtained are listed in Table 2.6. The durations vary between 25 minutes for the outflow 30 l/s·ha$_{red}$ and 360 minutes for the outflow 5 l/s·ha$_{red}$.

4 CONCLUSIONS

The best design of detention basins is done by using historical storms.

When using a design storm it was found important that the design storm has a total rainfall volume that is correct. This is especially pronounced when basins with small outflows are designed. For large catchments and for larger outflows the time variation of rainfall and the runoff calculation is also of significance.

The resulting basin volumes do not seem to be much influenced by the maximum water depth but further investigations are necessary for large (>5 m) and small depth (<1 m).

The I-D-F design storm and the Chicago design storm underestimated the detention basin volumes due to too small total volumes. Especially, the underestimations for the I-D-F design storm were significant and can not be ignored.

The use of the Sifalda design storm resulted in overestimated basin volumes. The Sifalda storm can be improved by using part I instead of part III for the estimations of the volumes, or by using both part I and part III.

The use of the traditional design method gave significant underestimations of the basin volumes. The underestimations can be compensated for by increase of the rainfall intensities, or increase of the basin volumes.

For practical applications the following is recommended when designing detention basins:

* A runoff model should be used for simulation of the inflow to the basins.

* Historical storms should be used if there is such data available.

* The Chicago design storm or the improved Sifalda design storm can be used if historical storms are not available.

* The Intensity-Duration-Frequency design storm should not be used.

* When using the traditional method for design the resulting basin volumes must be increased by 5-20% depending on the permitted outflow.

The investigation in this report is based on point precipitation. Therefore, the result is limited to small catchments. For larger basins the use of point precipitation data will result in overestimation of the basin volumes. Further research is necessary concerning the spatial variation of rainfall.

The conclusions are valid for catchments with runoff from impermeable areas only. For catchments with runoff from permeable areas it is important that the total rainfall volumes and the rainfall temporal patterns are correct, otherwise, the over- and underestimations will be larger than for catchments with runoff from impermeable areas only.

APPENDIX

STATISTICAL DISTRIBUTIONS FOR CALCULATED BASIN VOLUMES

Plotting of Calculated Detention Basin Volumes for Historical Storms and Different Design Storms

After calculation of the basin volumes with the CTH-Model for the historical storms, the 36 largest basin volumes for each catchment and each outflow 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

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

where

y_i = plotting positions for the calculated basin volumes in increasing order.

N = number of treated basin volumes which are chosen as equal to the number of treated time periods, in this case 36 ½-year periods.

y_i is related to the return period through the relationship

$$y_i = \ln F \qquad \ldots.(I.2)$$

where

F = return period in number of treated time periods, in this case ½-year periods (in the figures, F is given in years).

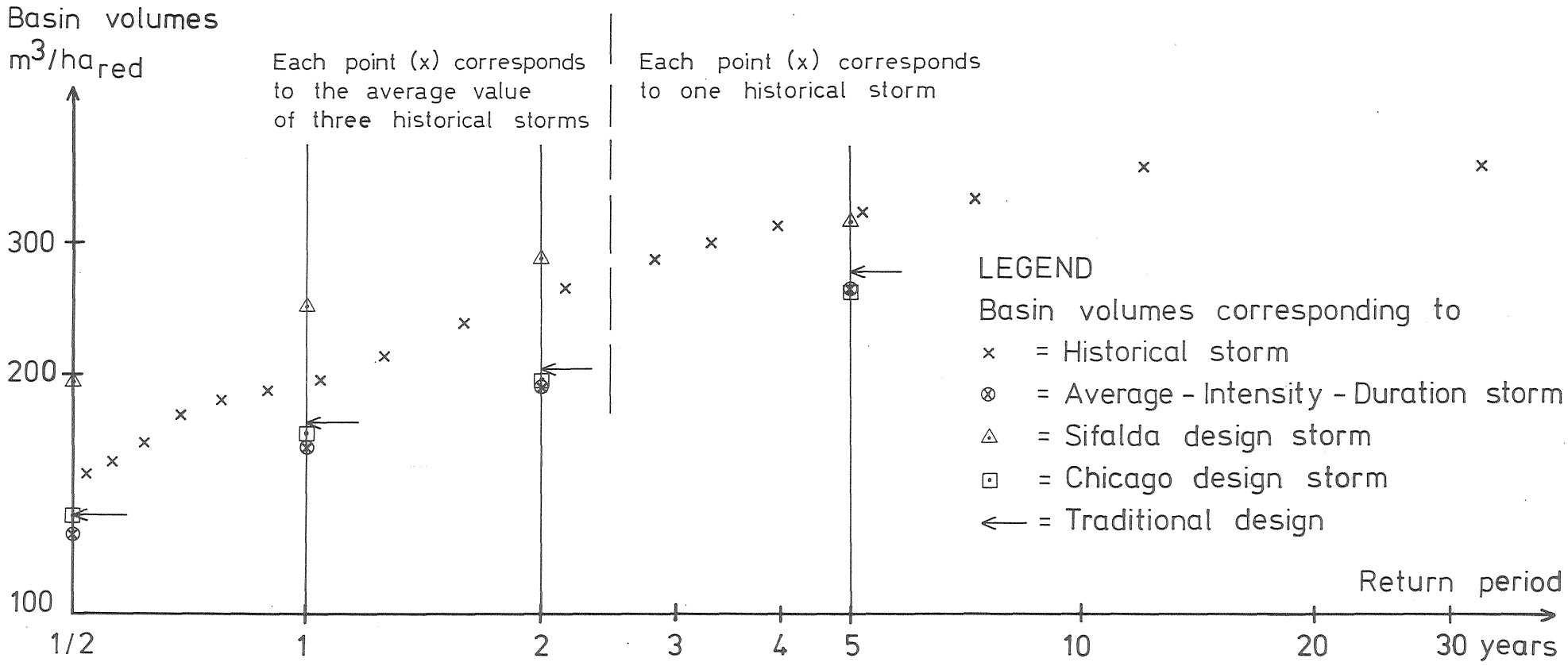
In order to make the plottings more clear, the 7 largest basin volumes were plotted as individual points and the remaining basin volumes were plotted as average values of 3 volumes and 3 y-values.

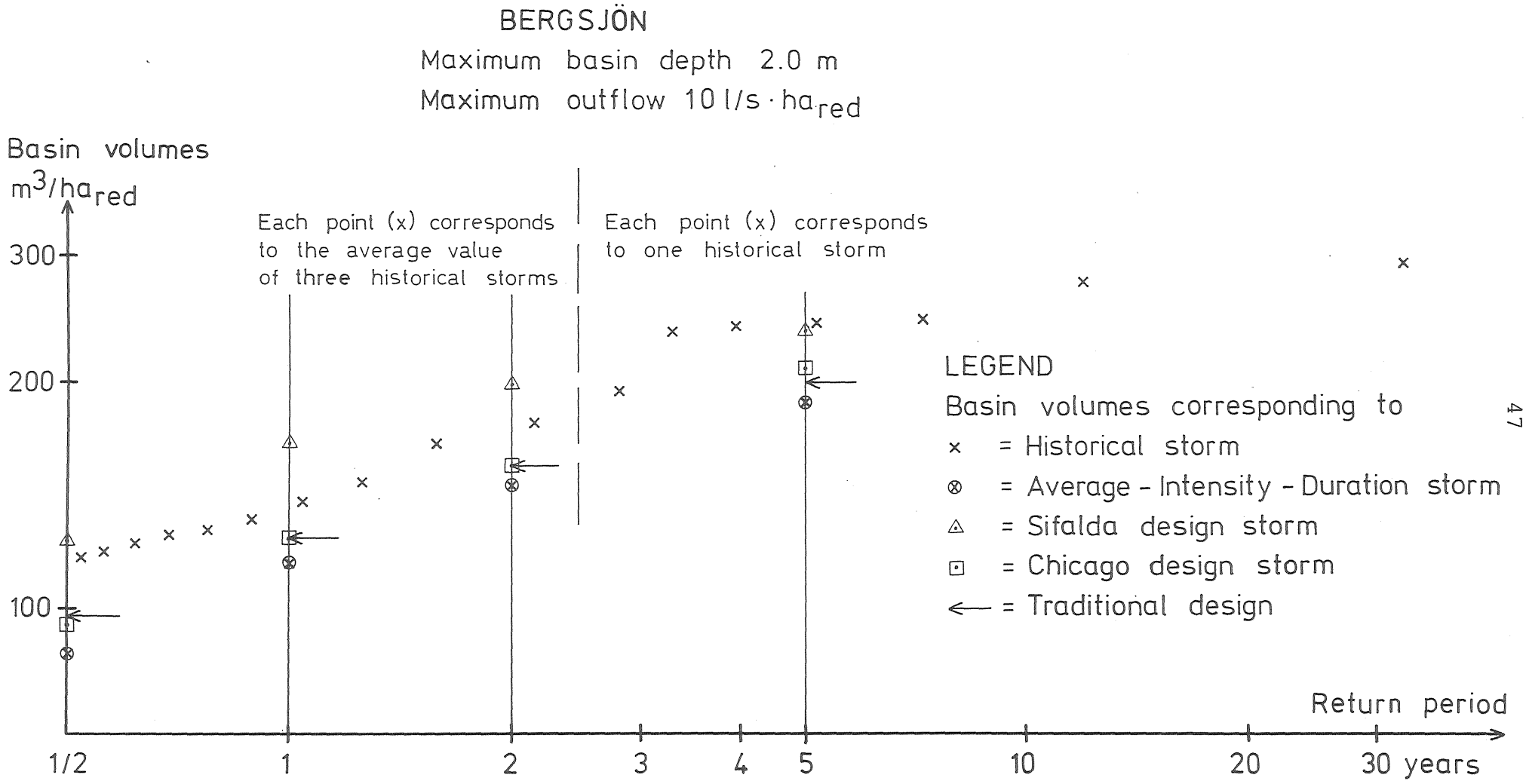
The basin volumes for the different design storms were plotted in the diagrams for the return periods ½, 1, 2, and 5 years.

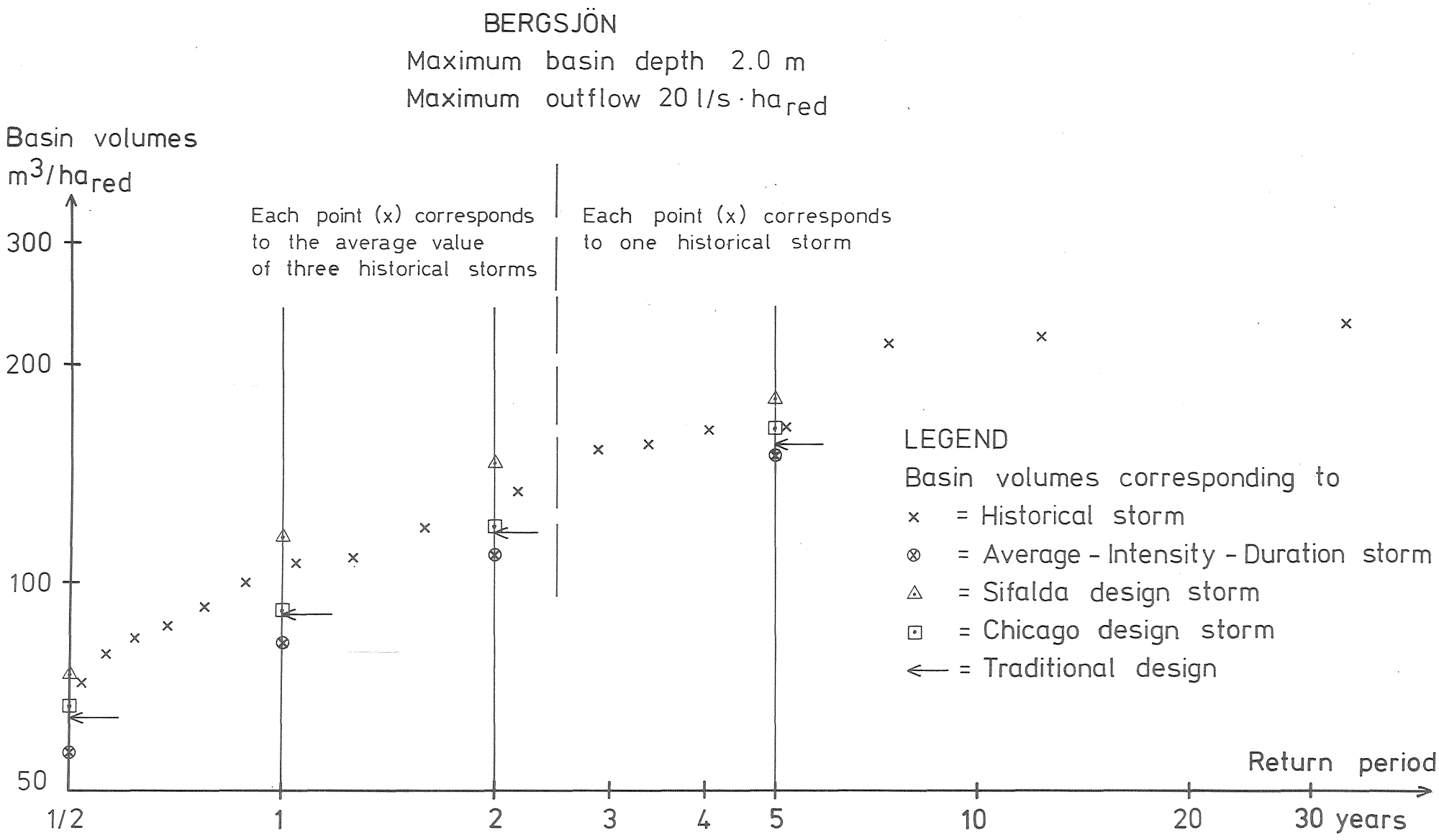
BERGSJÖN

Maximum basin depth 2.0 m

Maximum outflow 5 l/s · ha_{red}



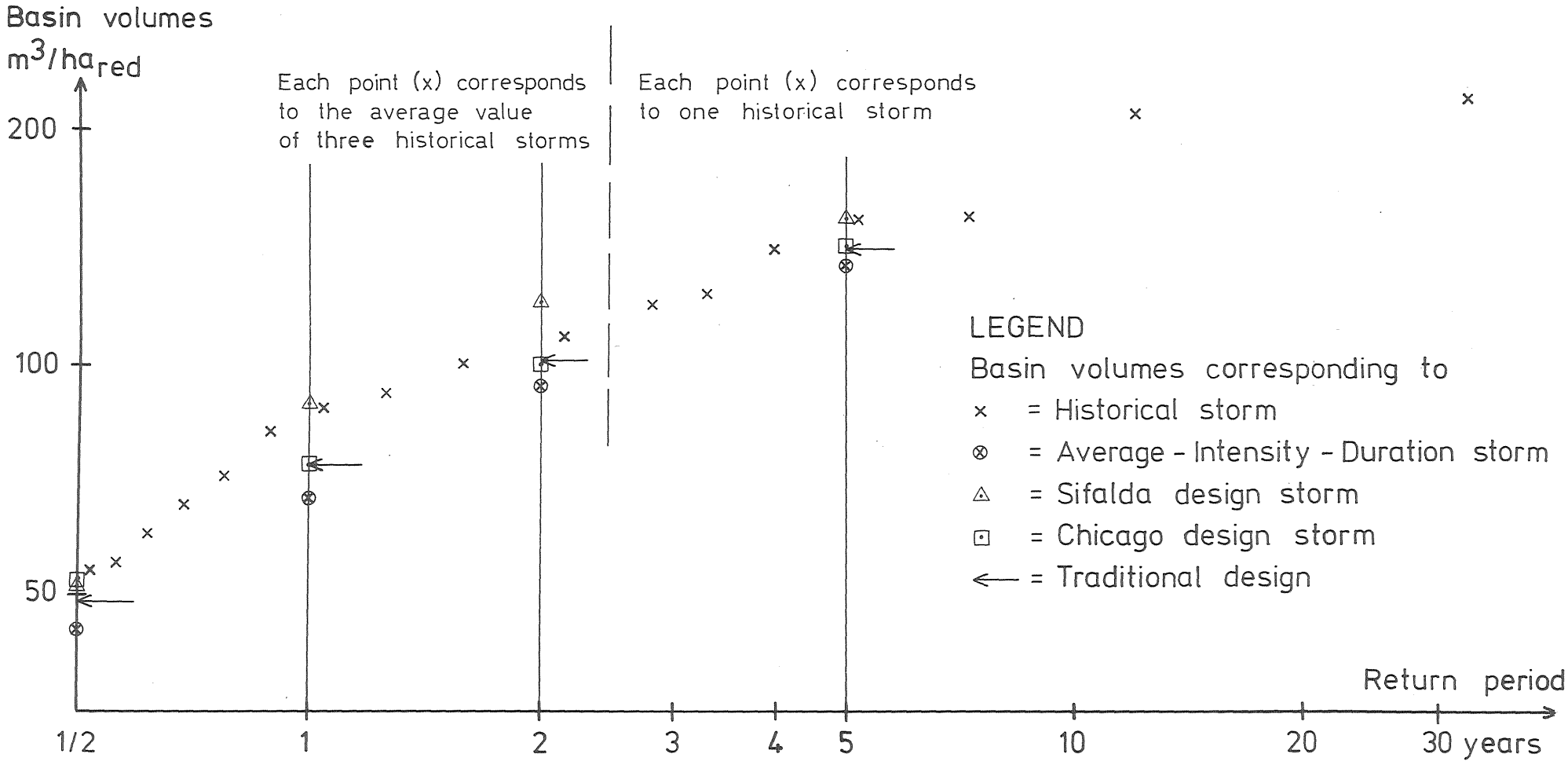

BERGSJÖN
Maximum basin depth 2.0 m
Maximum outflow 10 l/s · ha red
Basin volumes
m3/ha red
300
200
100
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years


BERGSJÖN
Maximum basin depth 2.0 m
Maximum outflow 20 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

BERGSJÖN

Maximum basin depth 2.0 m

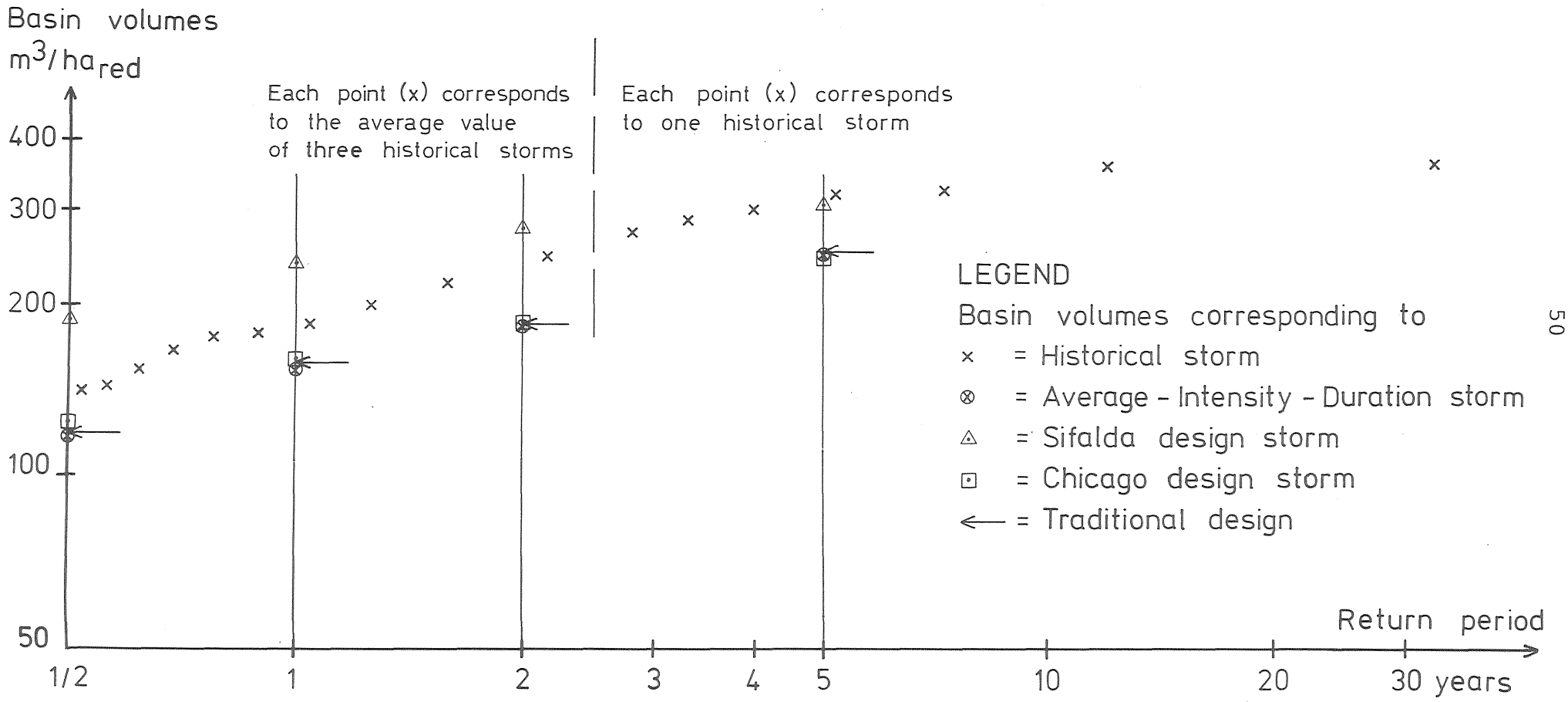
Maximum outflow 30 l/s · ha_{red}



LINKÖPING 1

Maximum basin depth 2.0 m

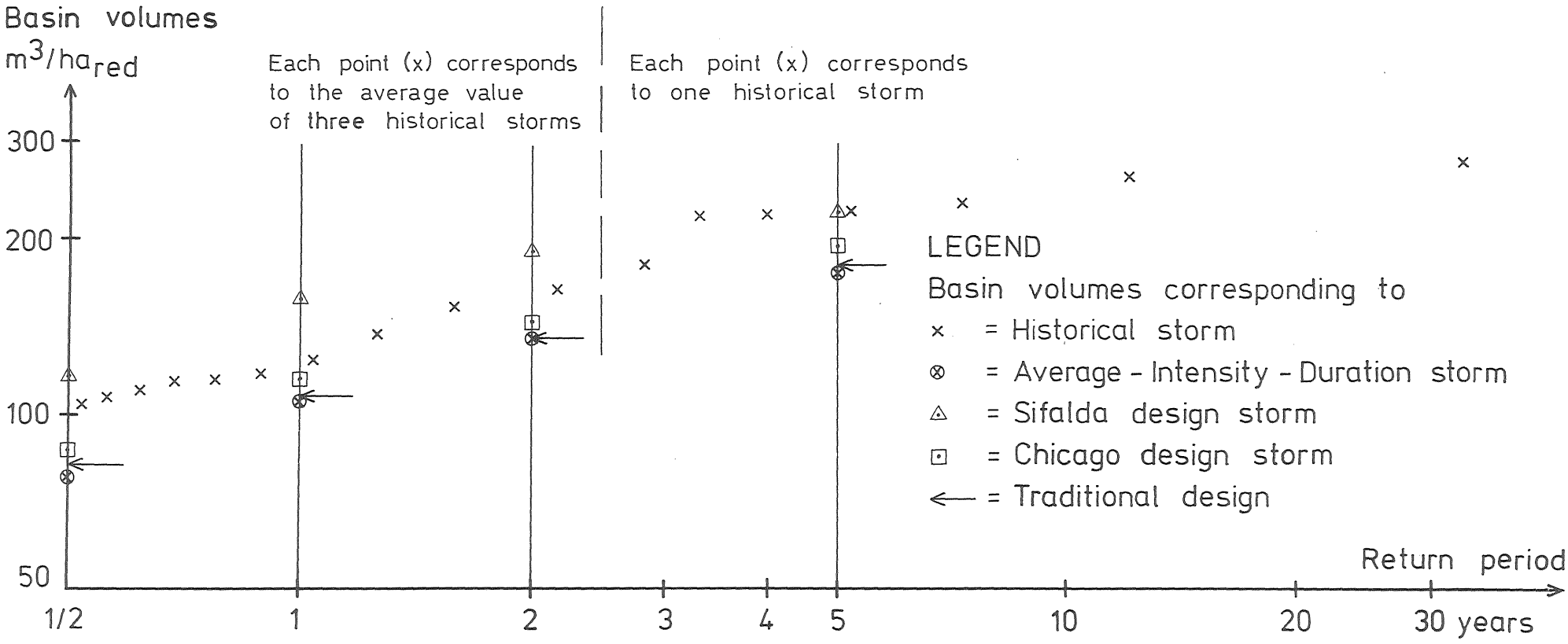
Maximum outflow 5 l/s · ha_{red}

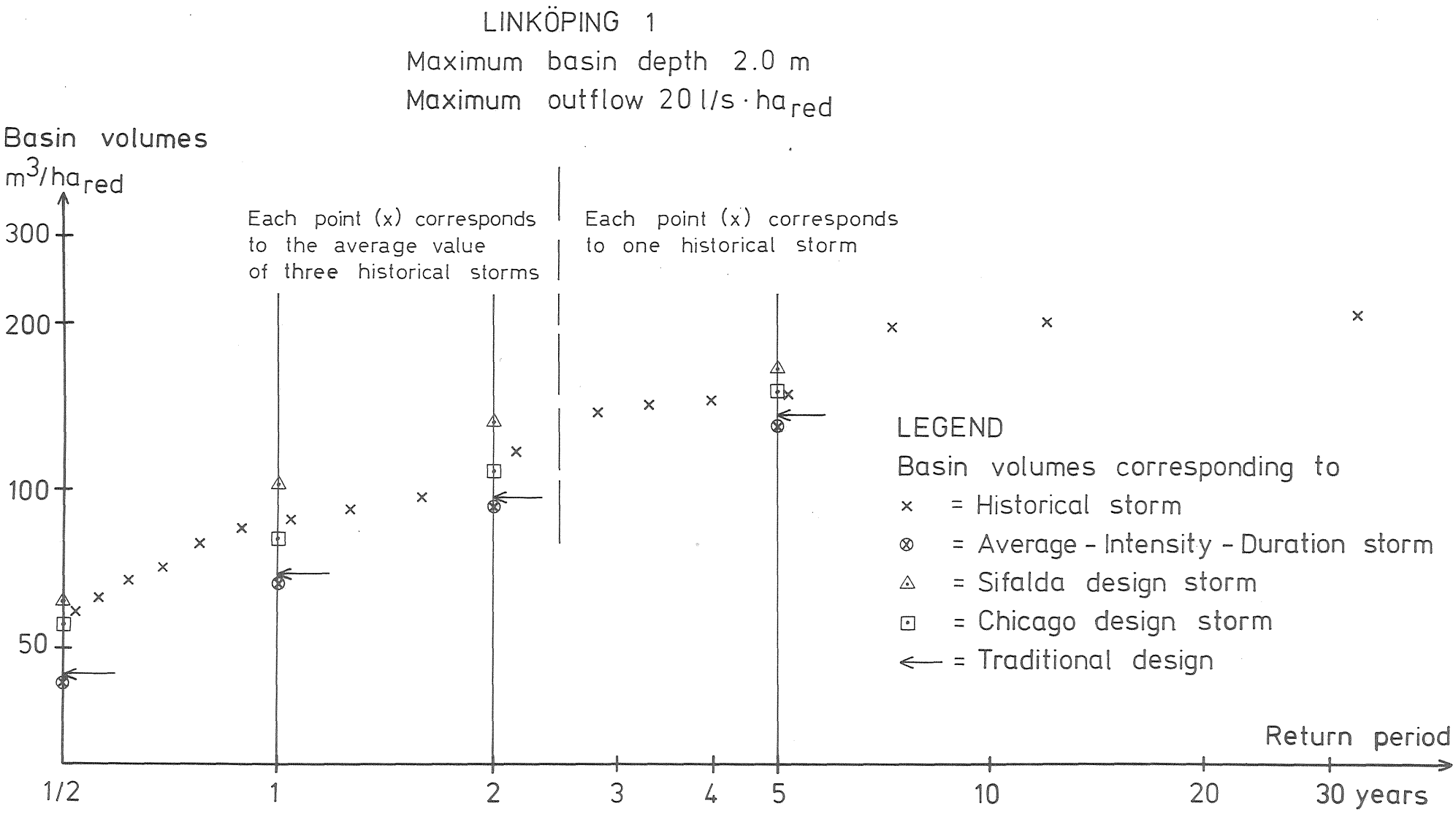


LINKÖPING 1

Maximum basin depth 2.0 m

Maximum outflow 10 l/s · ha_{red}




LINKÖPING 1
Maximum basin depth 2.0 m
Maximum outflow 20 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
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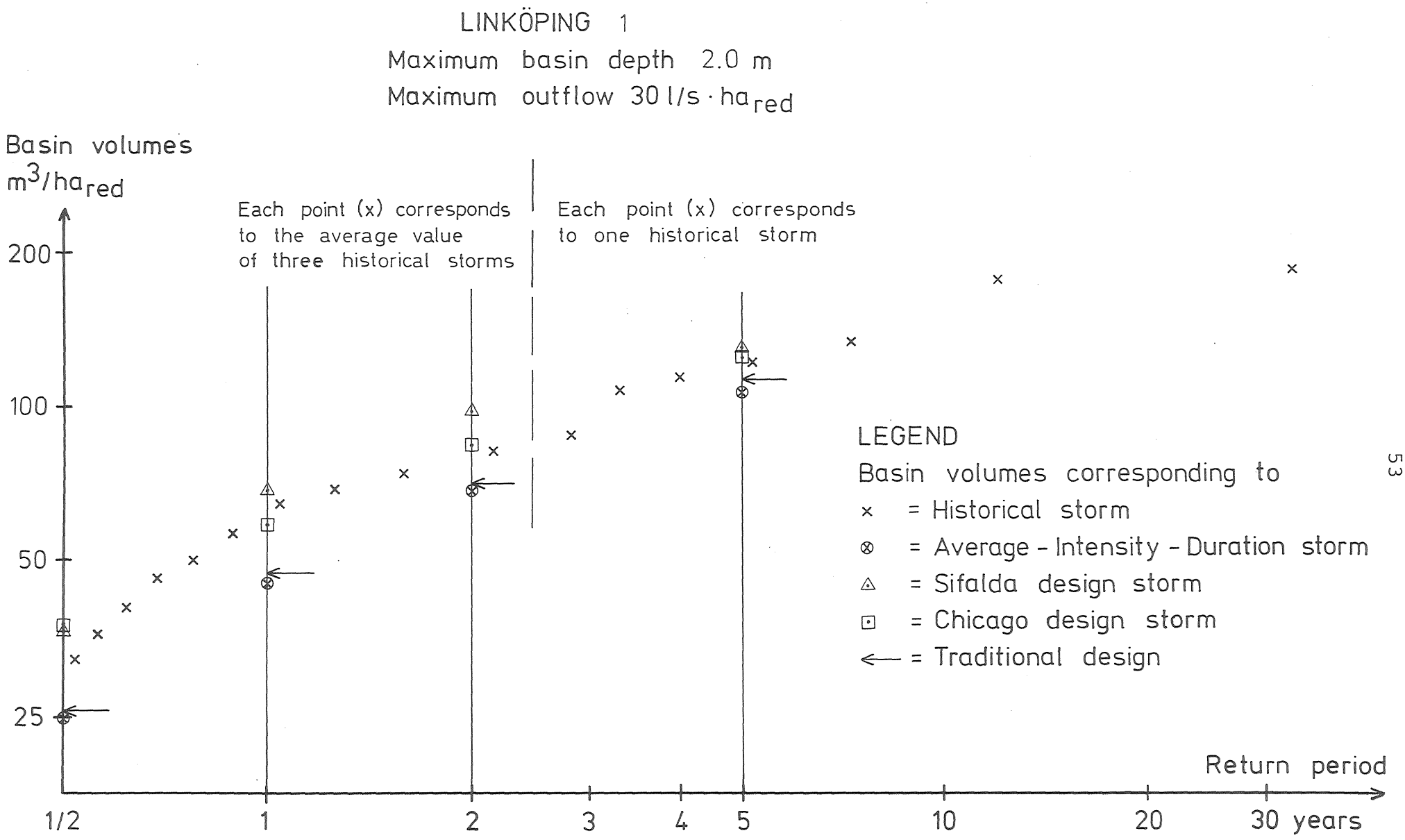
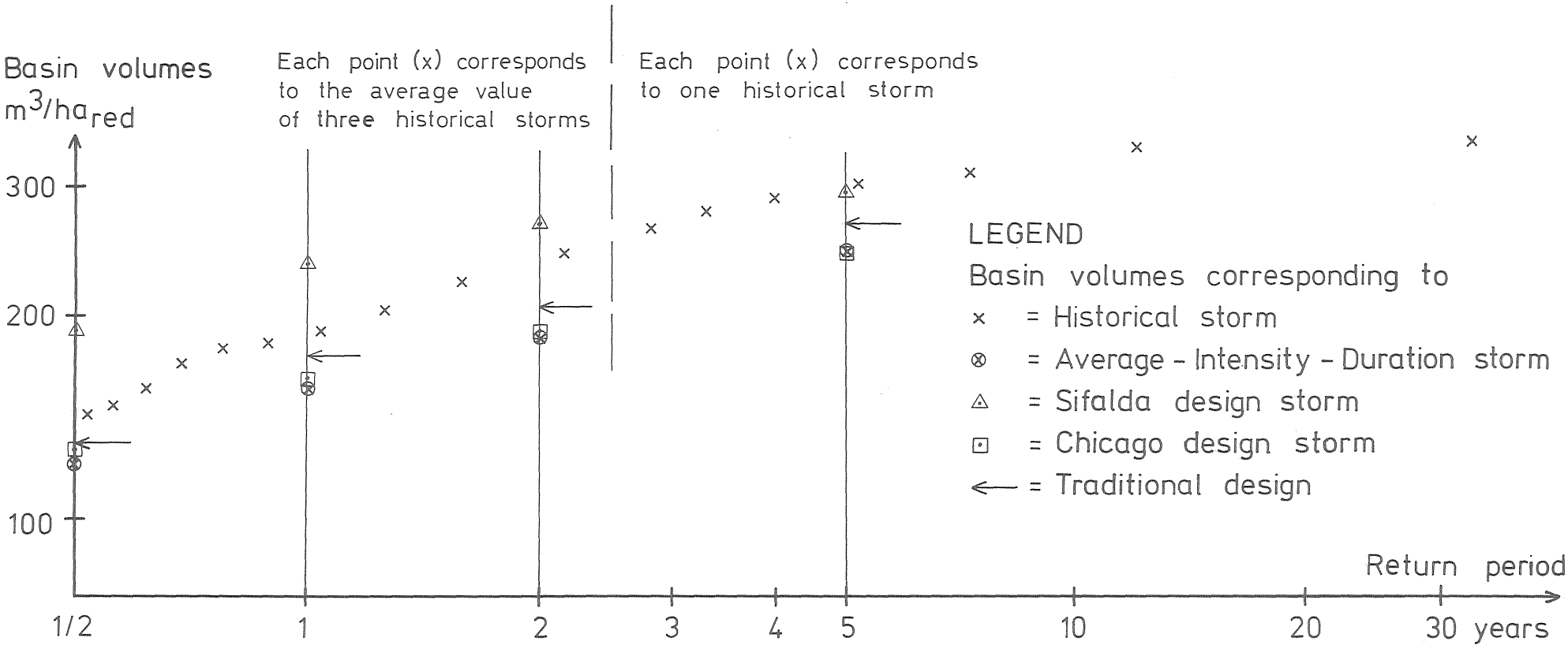
10
20
30 years

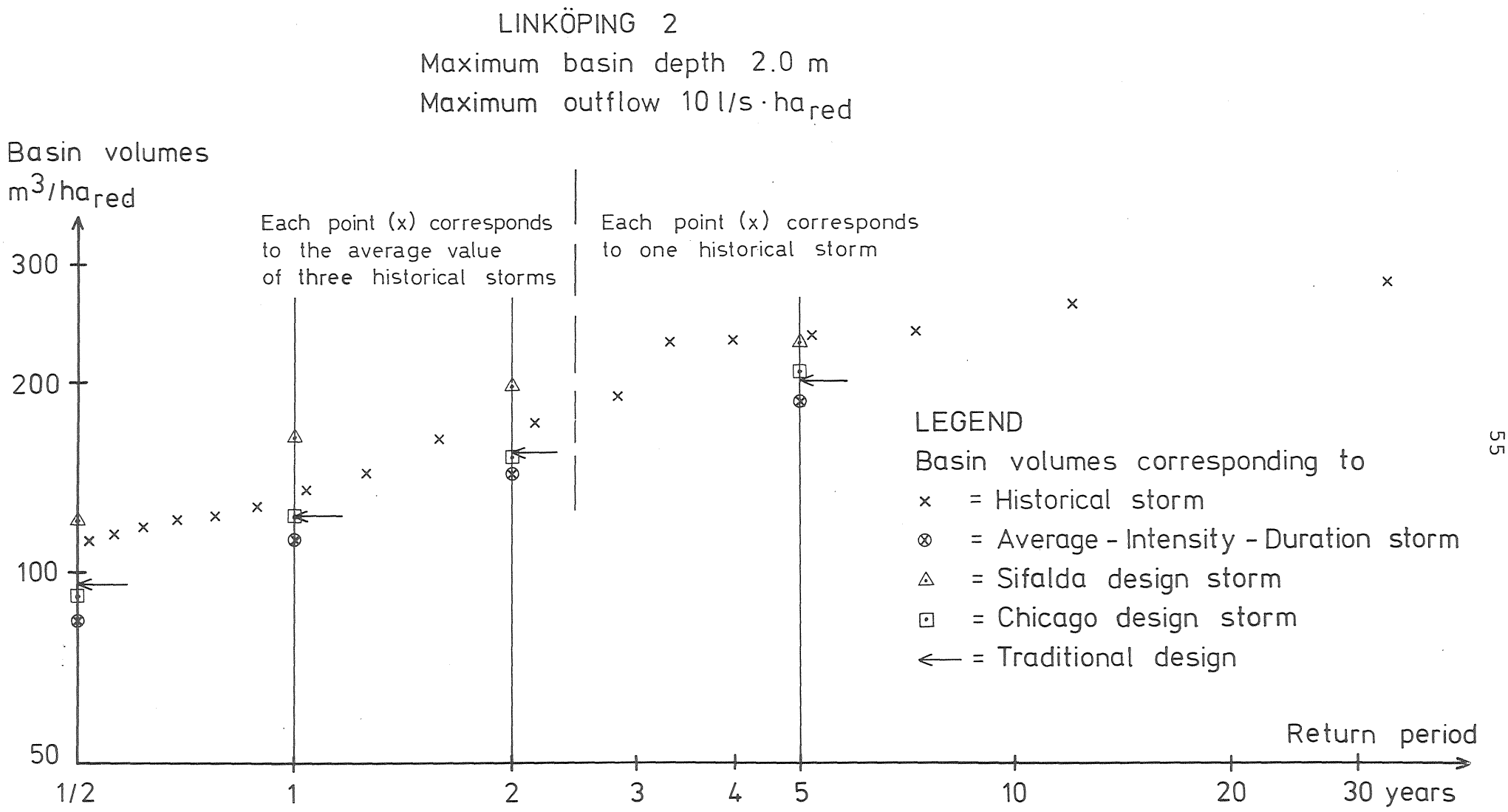

LINKÖPING 1
Maximum basin depth 2.0 m
Maximum outflow 30 l/s · ha red
Basin volumes
m3/ha red
200
100
50
25
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

LINKÖPING 2

Maximum basin depth 2.0 m

Maximum outflow 5 l/s · ha_{red}




LINKÖPING 2
Maximum basin depth 2.0 m
Maximum outflow 10 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
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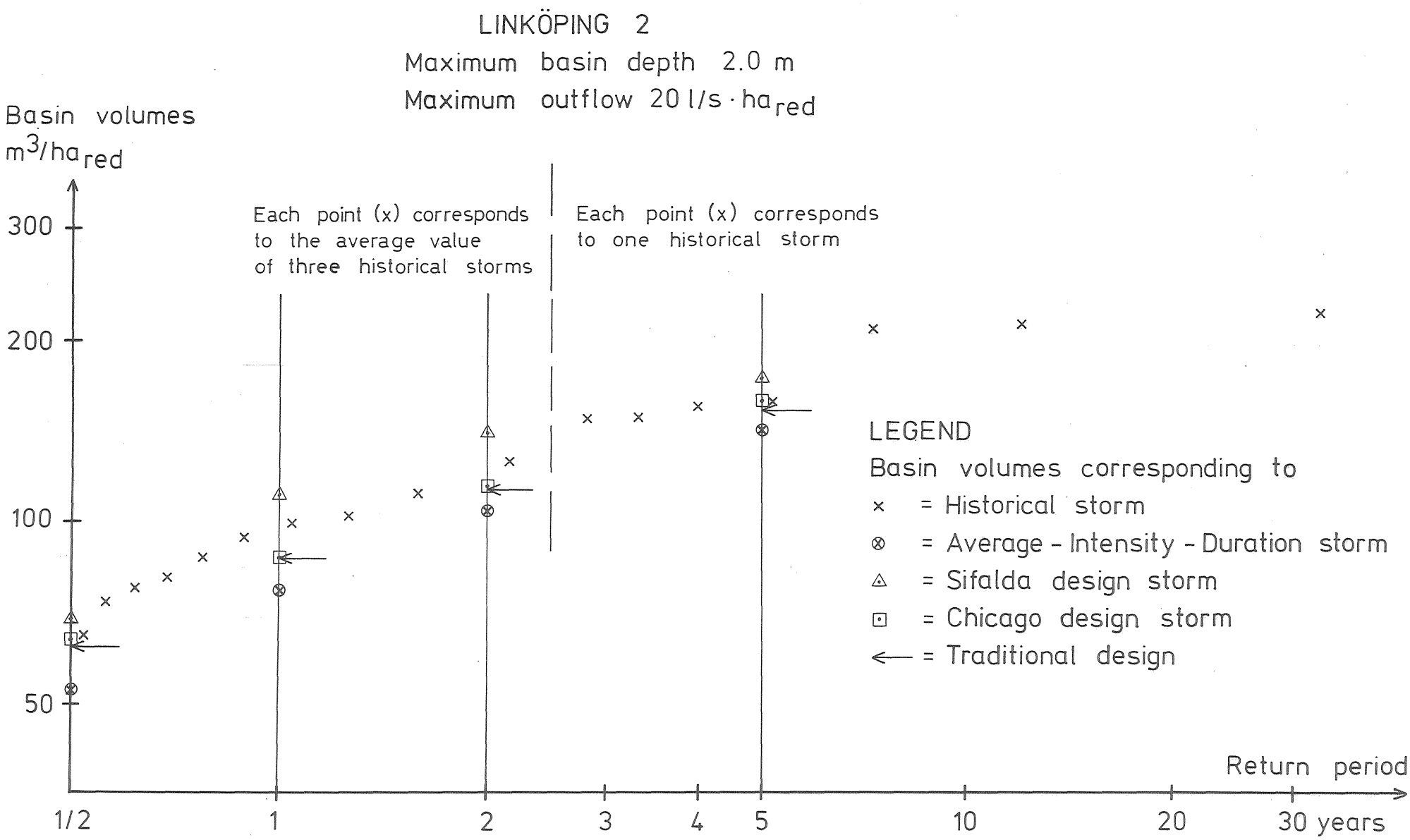
5
10
20
30 years


LINKÖPING 2
Maximum basin depth 2.0 m
Maximum outflow 20 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
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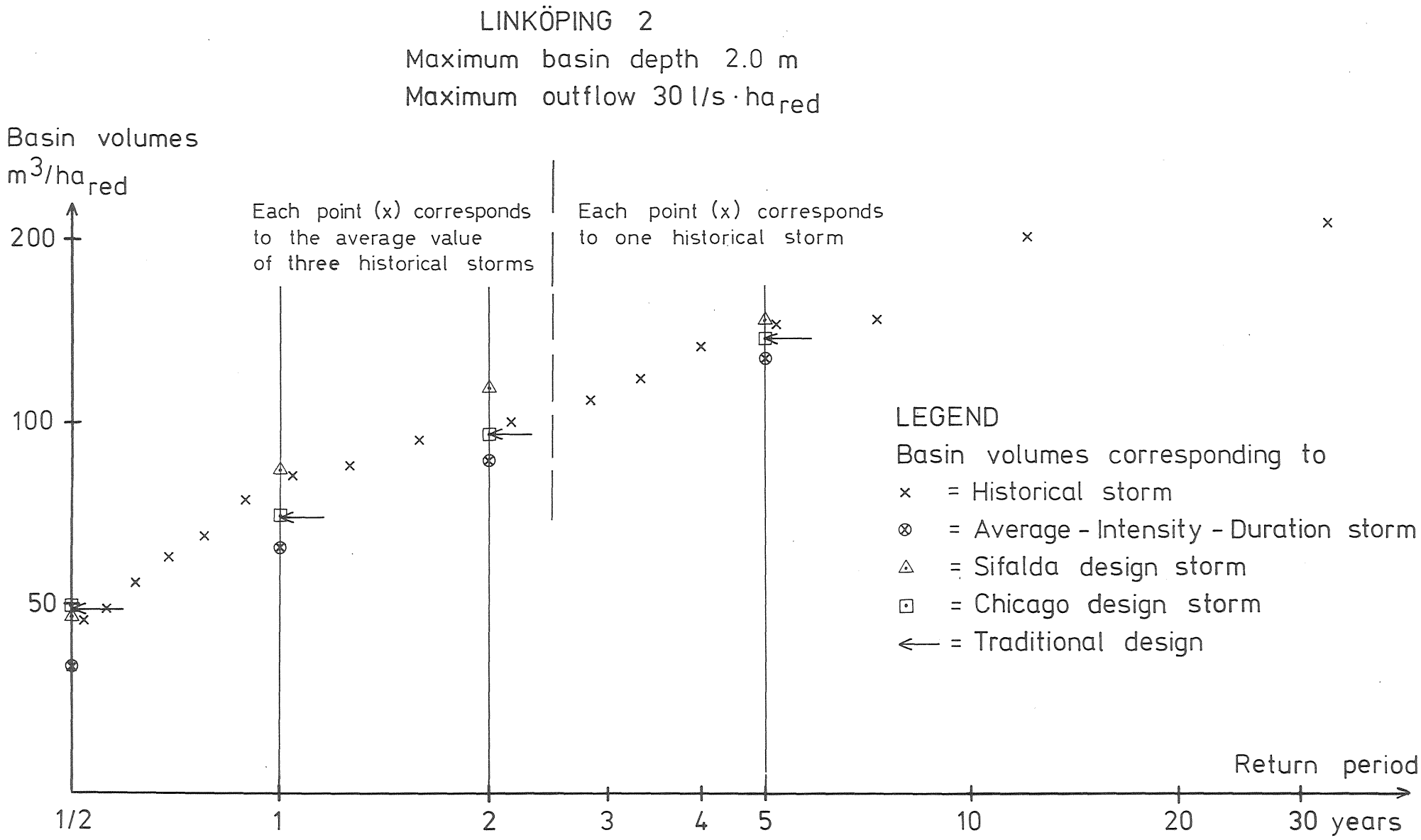
5
10
20
30 years


LINKÖPING 2
Maximum basin depth 2.0 m
Maximum outflow 30 l/s · ha red
Basin volumes
m3/ha red
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
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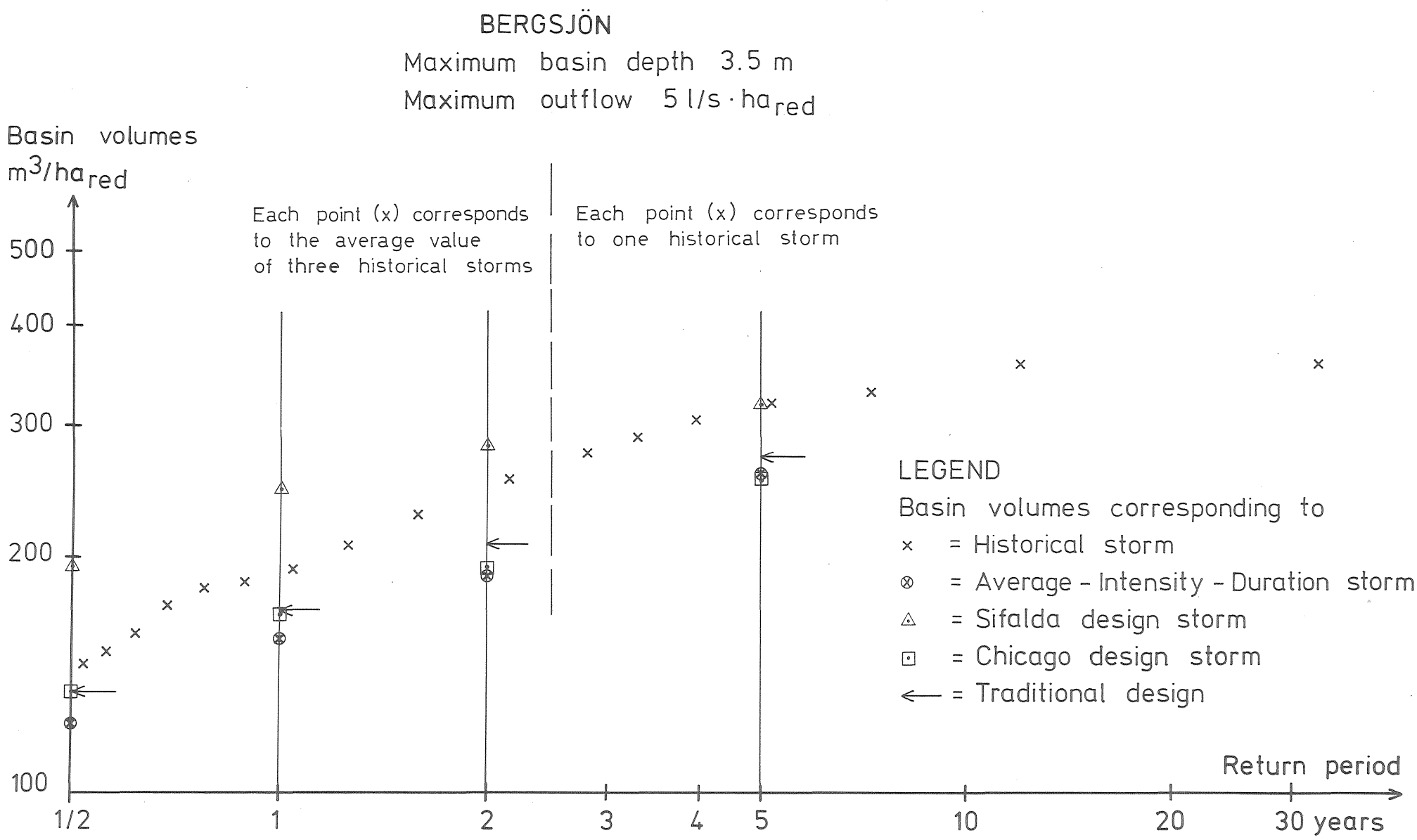
20
30 years


BERGSJÖN
Maximum basin depth 3.5 m
Maximum outflow 5 l/s · ha red
Basin volumes
m3/ha red
500
400
300
200
100
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
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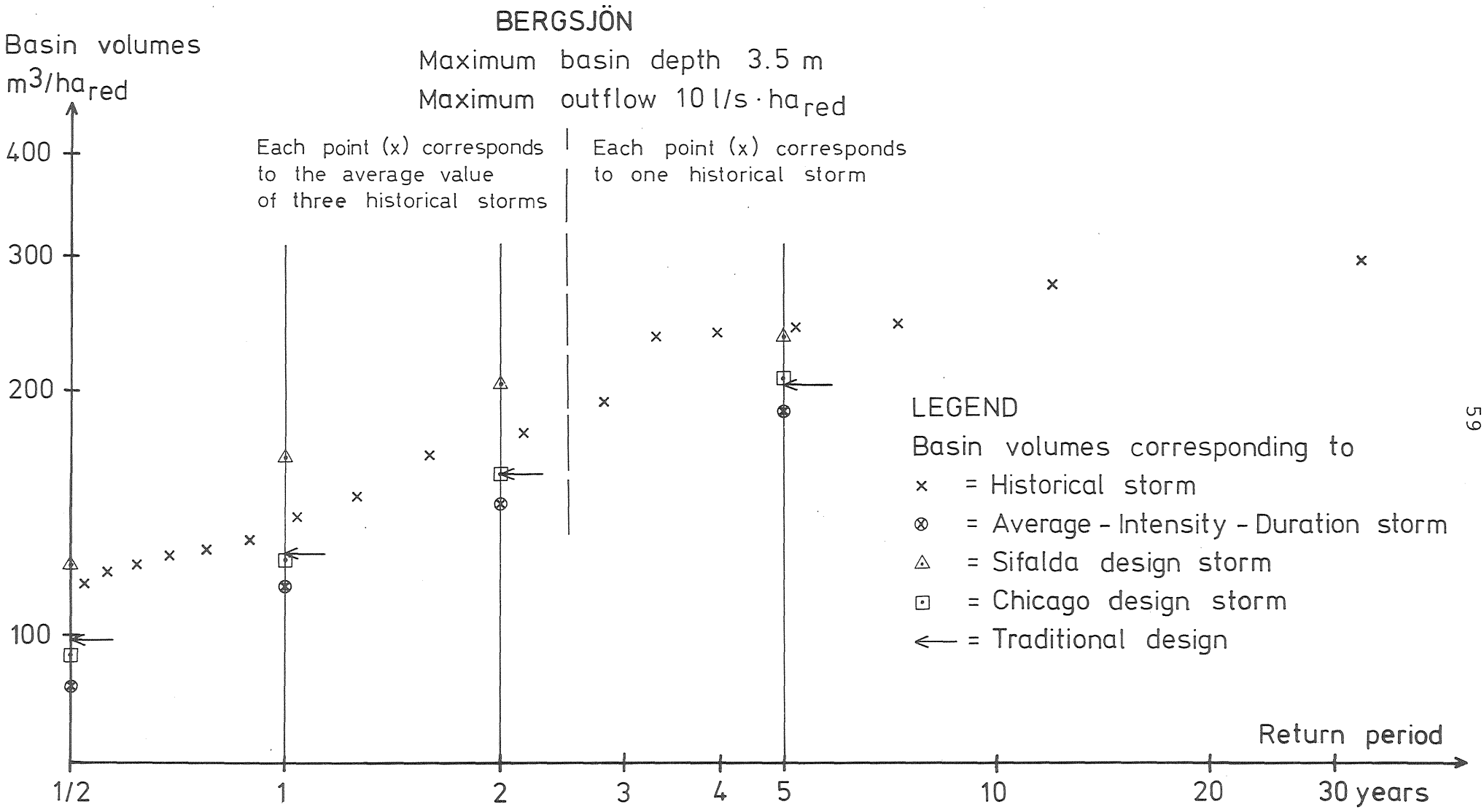
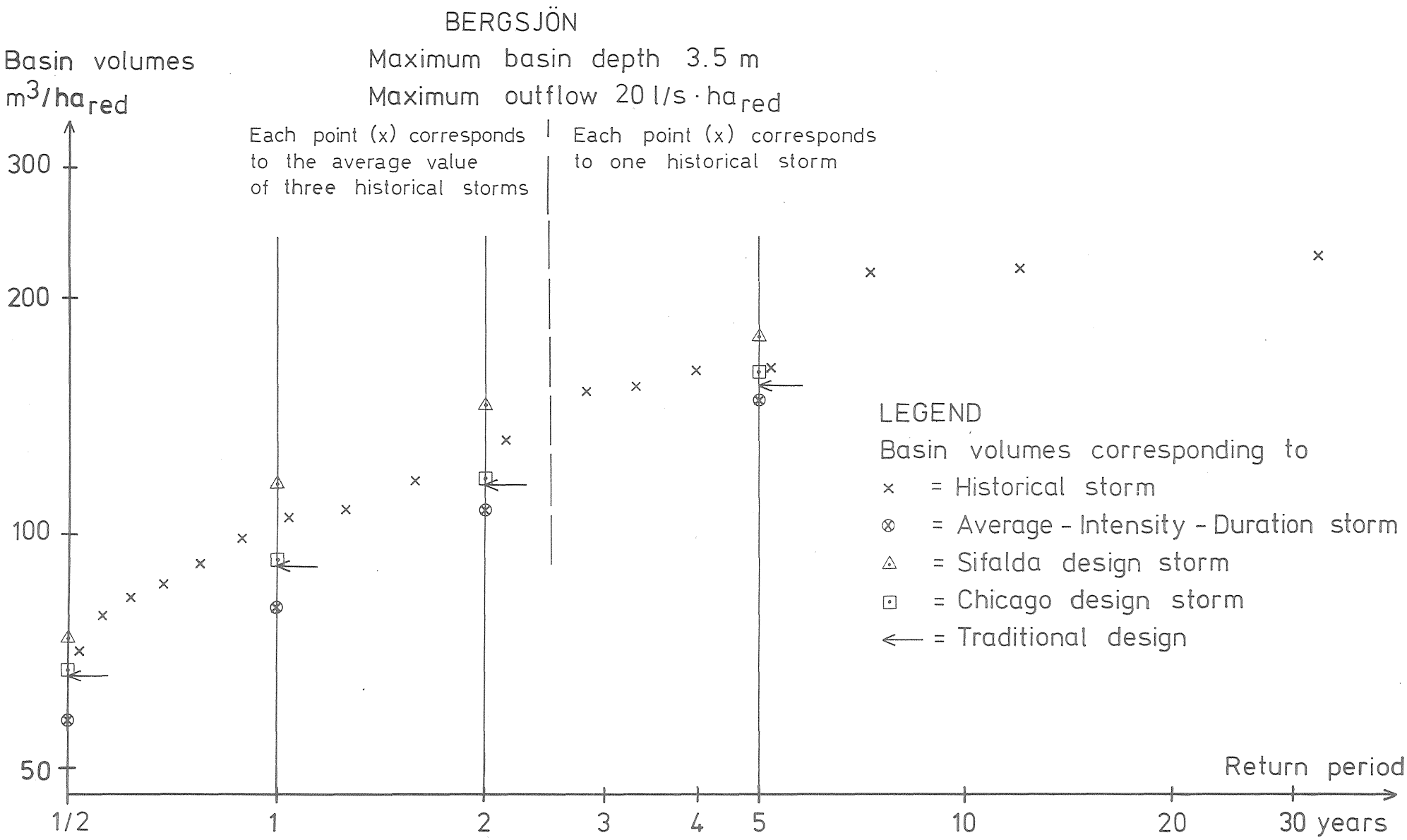
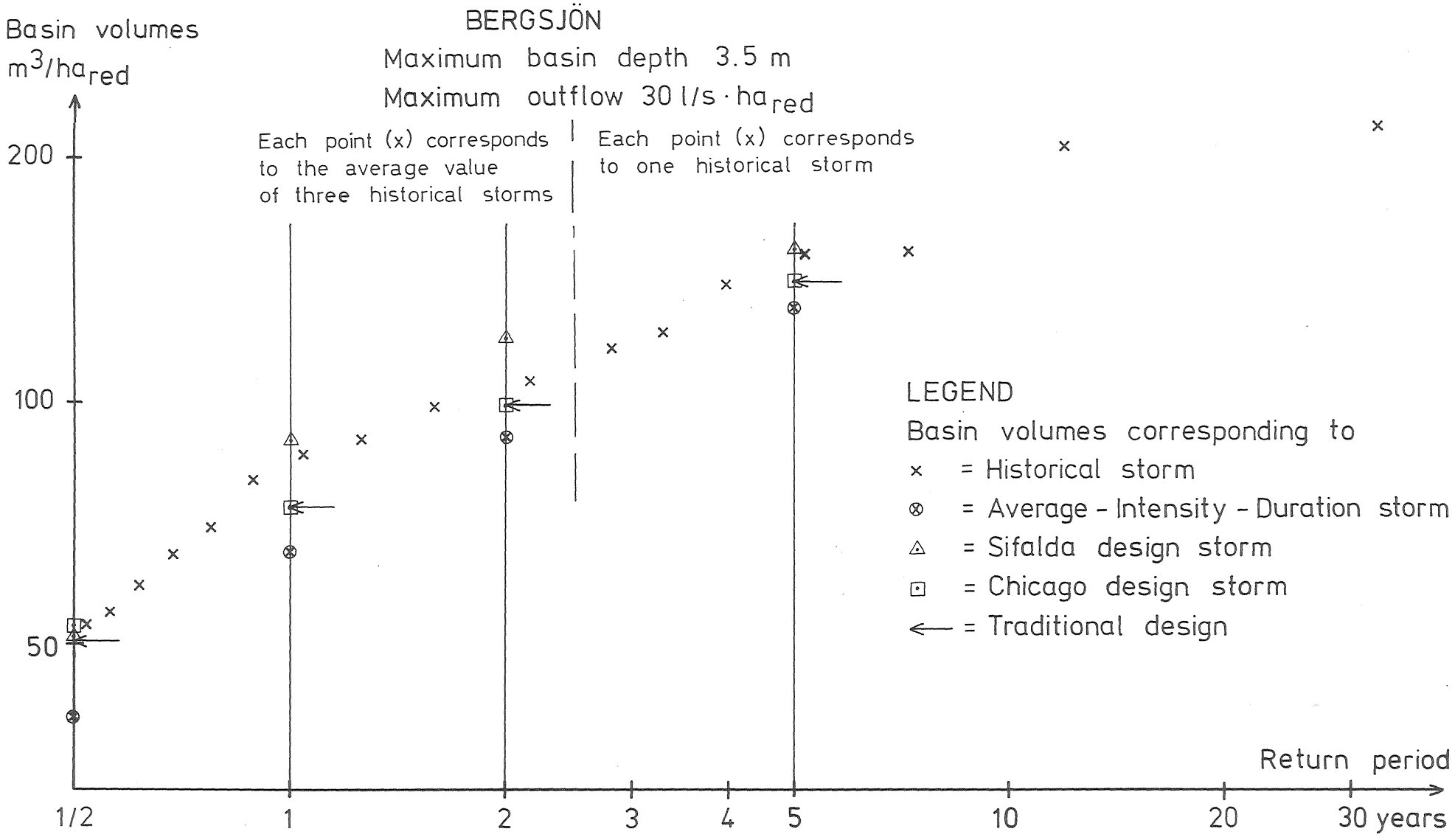
10
20
30 years


BERGSJÖN
Maximum basin depth 3.5 m
Maximum outflow 10 l/s · ha red
Basin volumes
m3/ha red
400
300
200
100
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

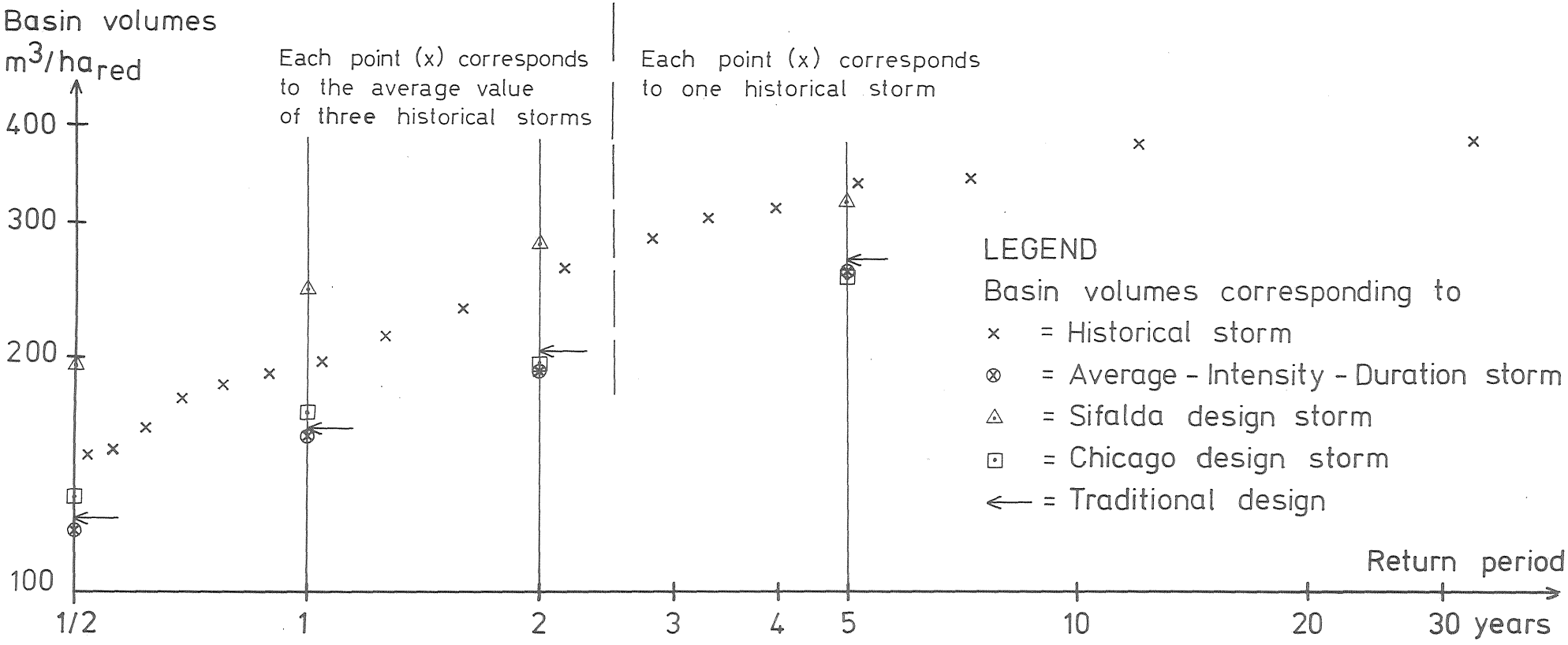

BERGSJÖN
Maximum basin depth 3.5 m
Maximum outflow 20 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

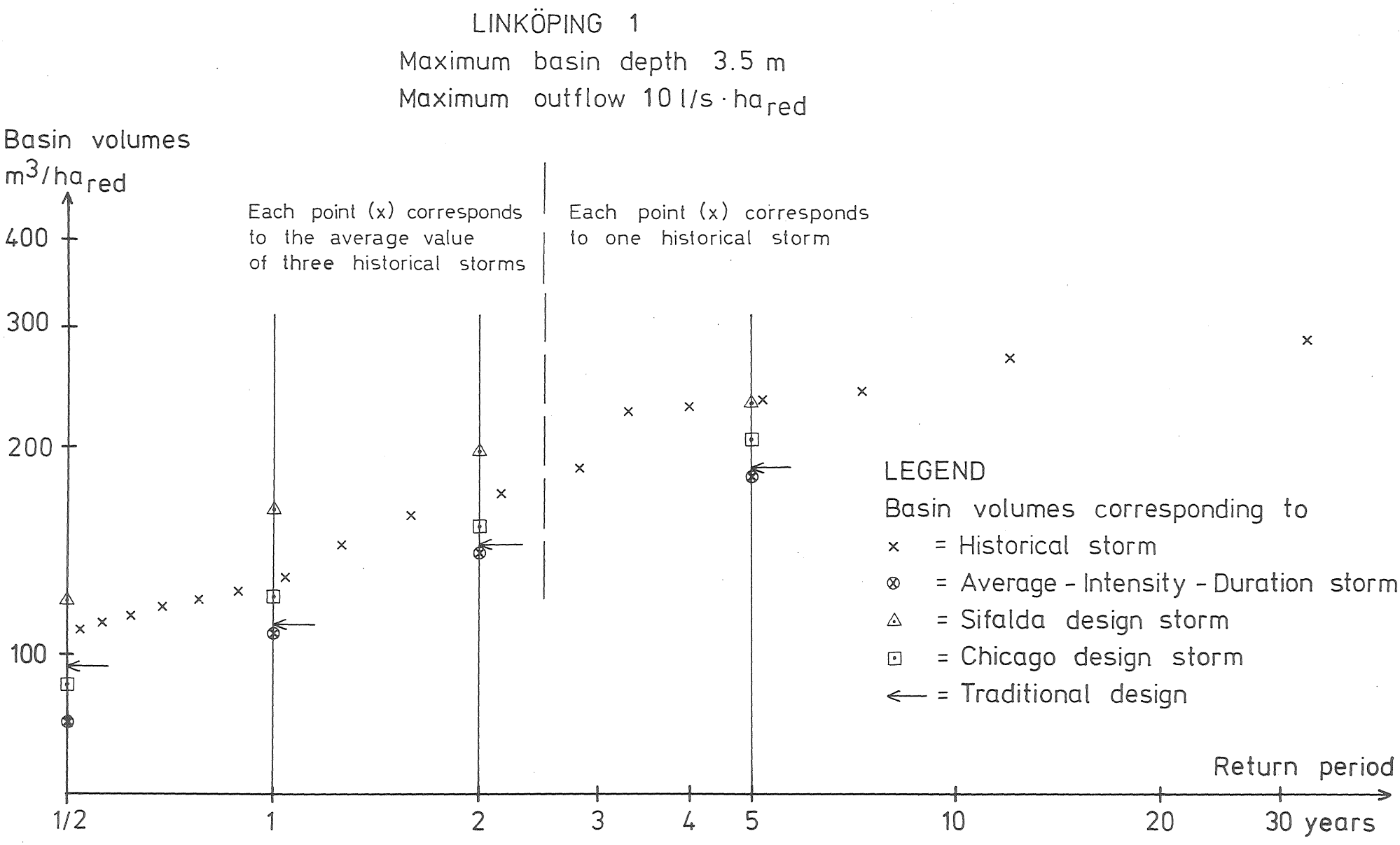

BERGSJÖN
Maximum basin depth 3.5 m
Maximum outflow 30 l/s · ha red
Basin volumes
m3/ha red
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

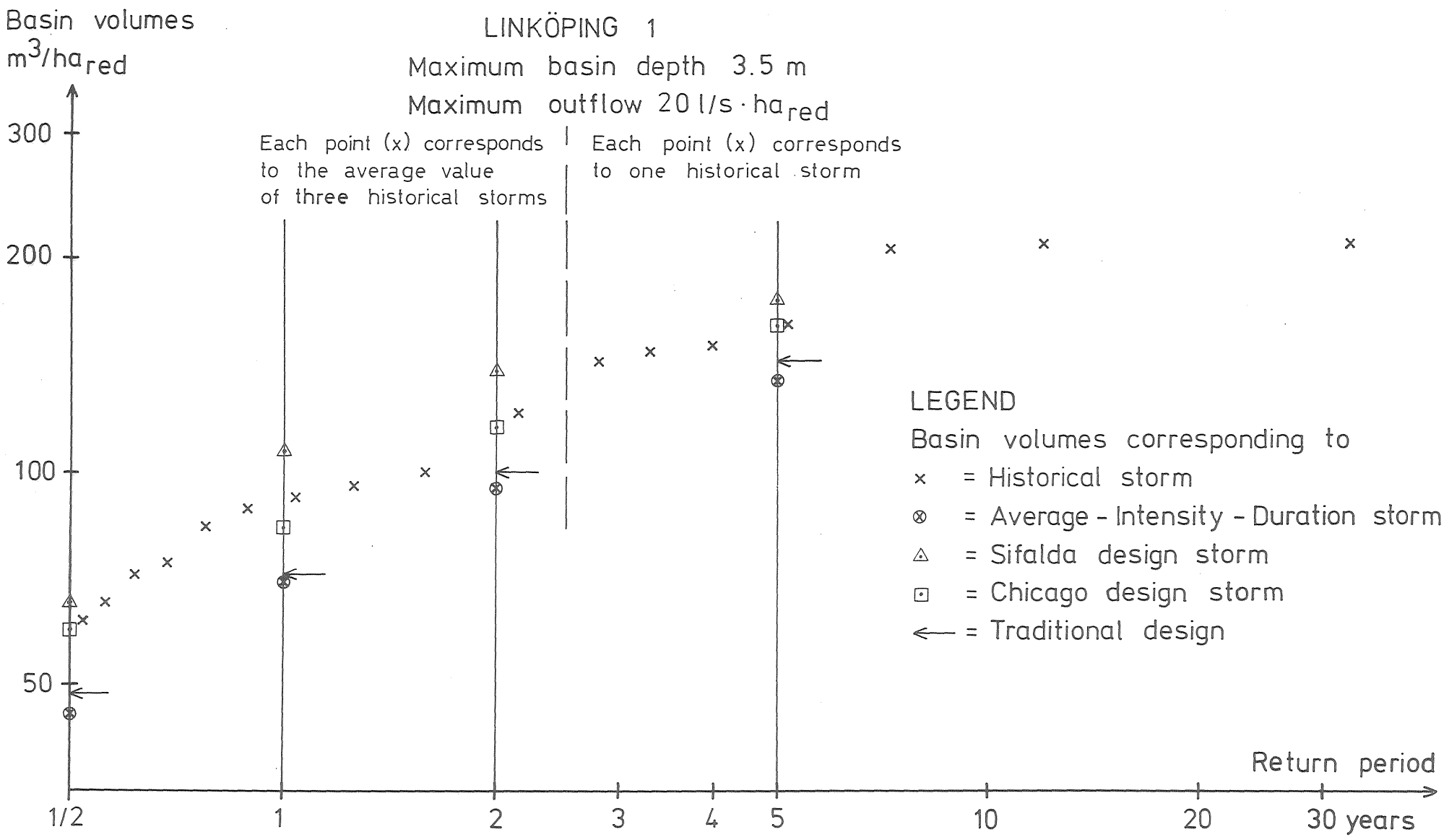
LINKÖPING 1

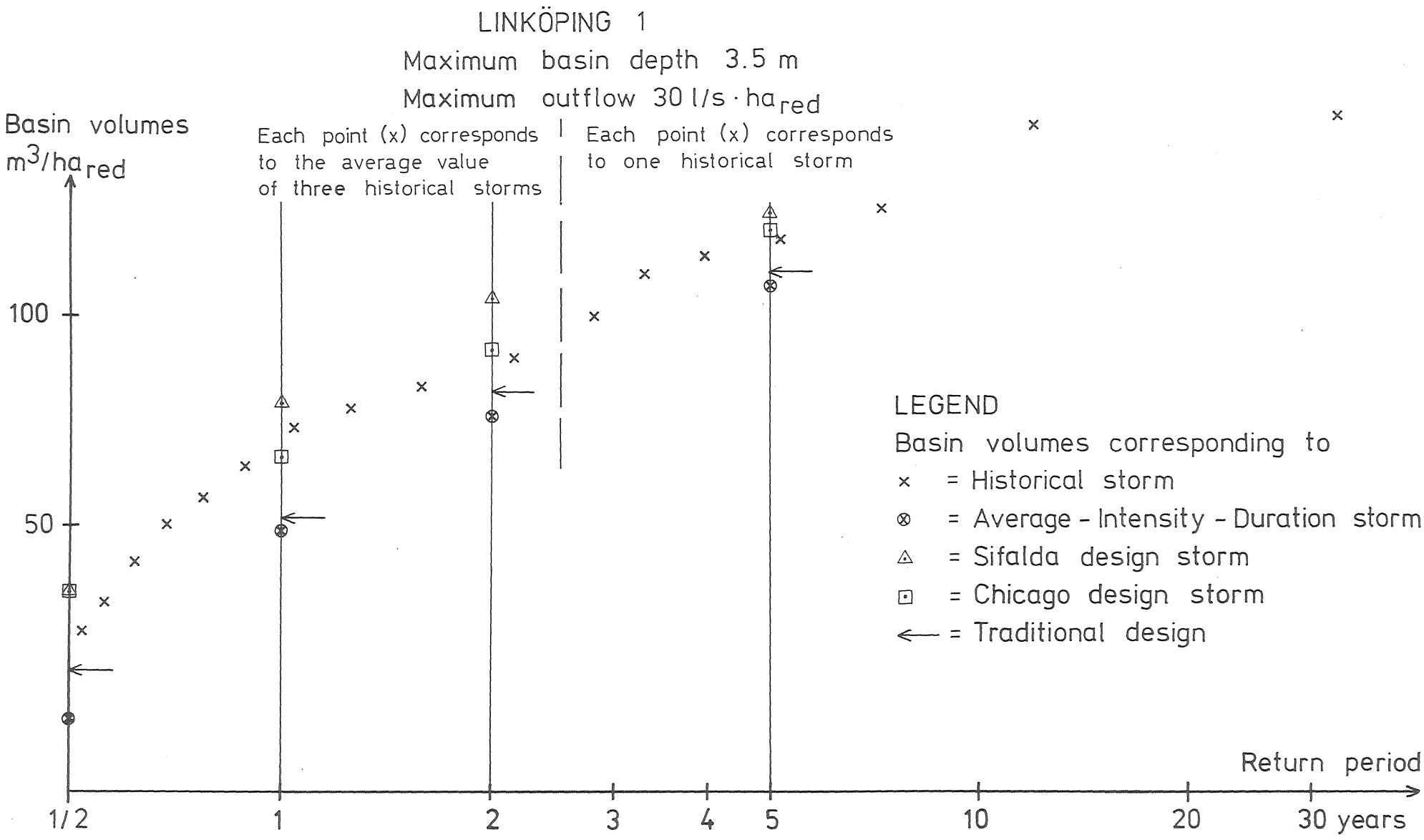
Maximum basin depth 3.5 m

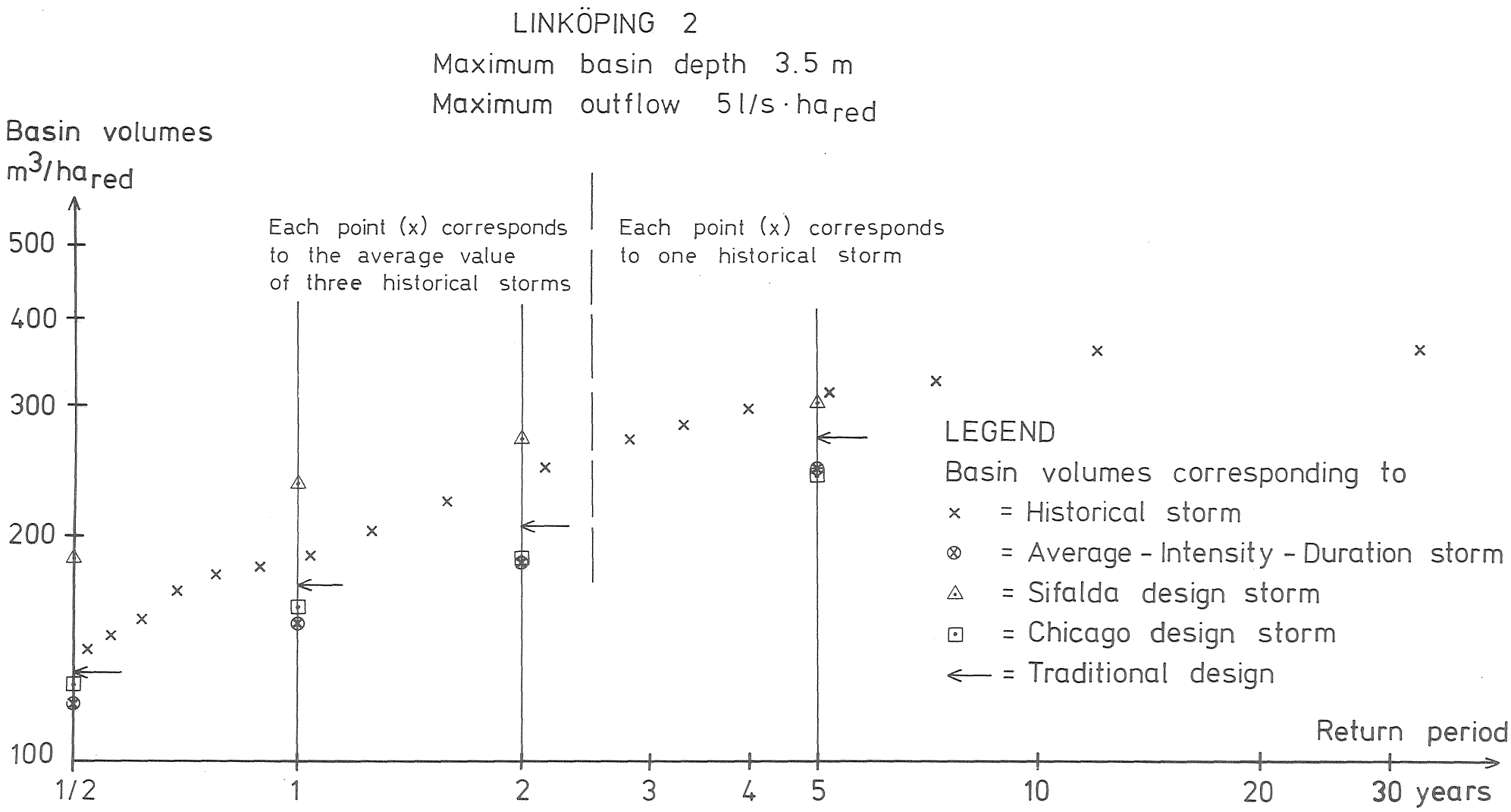
Maximum outflow 5 l/s · ha_{red}

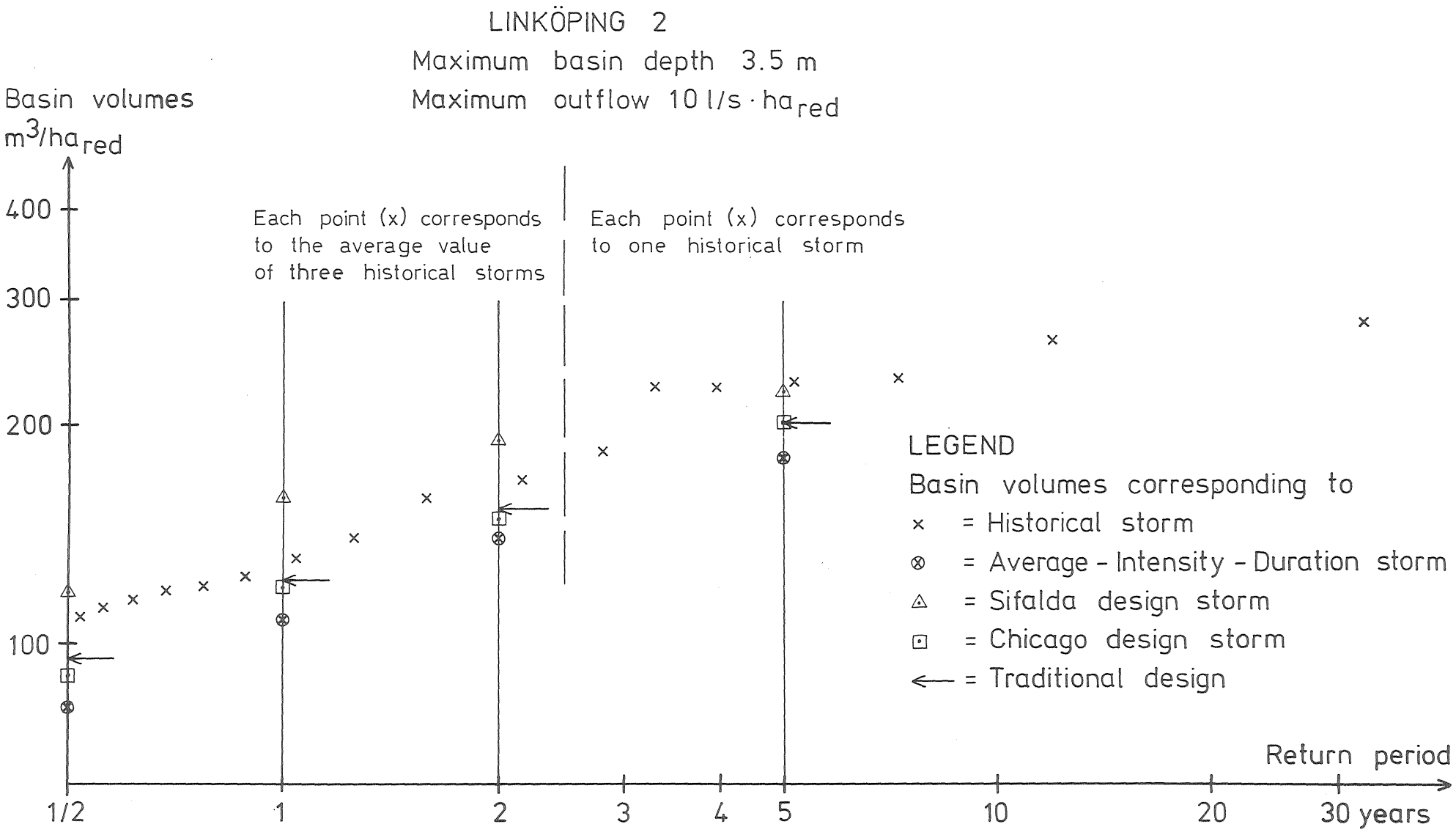


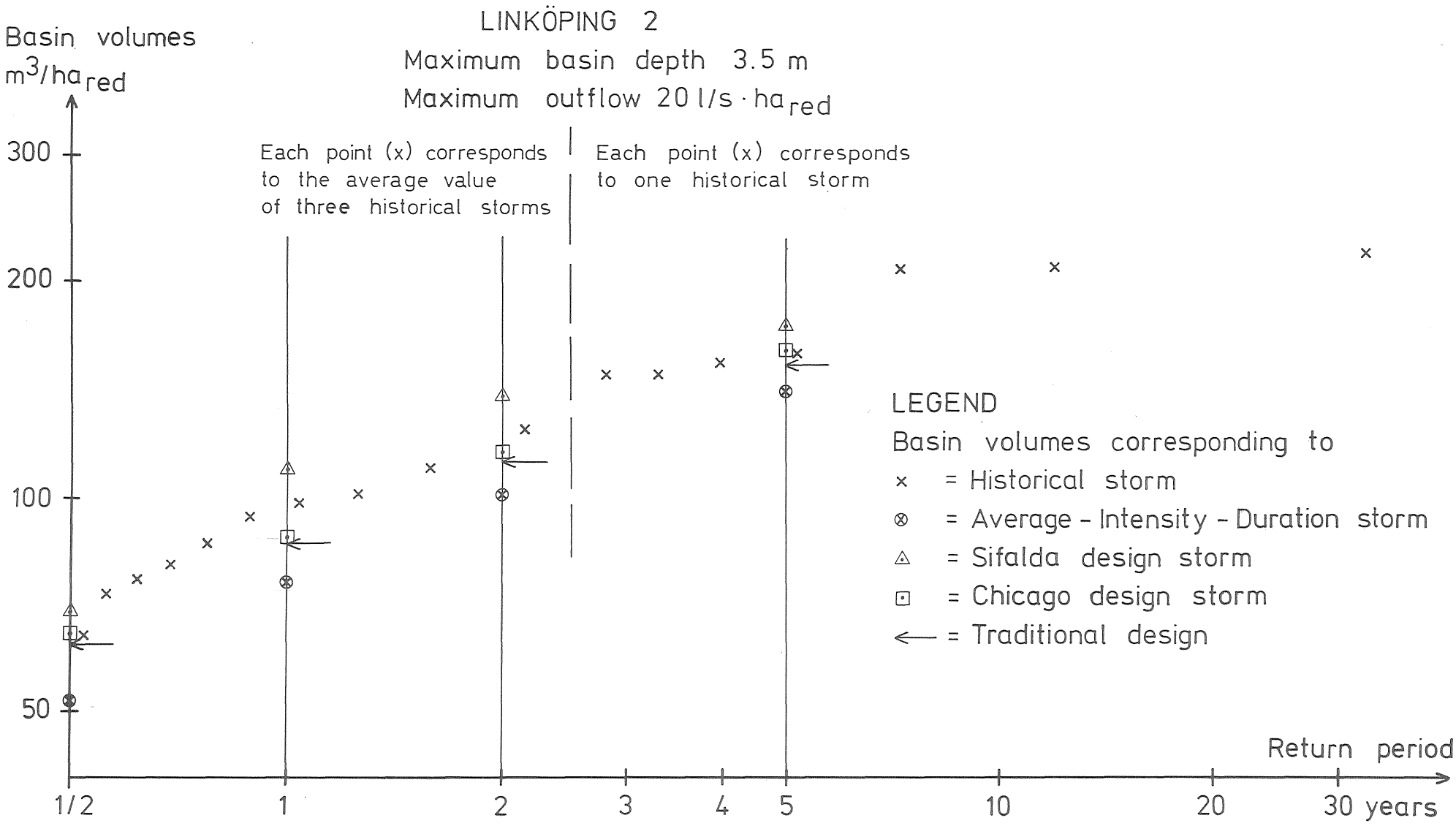

LINKÖPING 1
Maximum basin depth 3.5 m
Maximum outflow 10 l/s · ha red
Basin volumes
m3/ha red
400
300
200
100
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

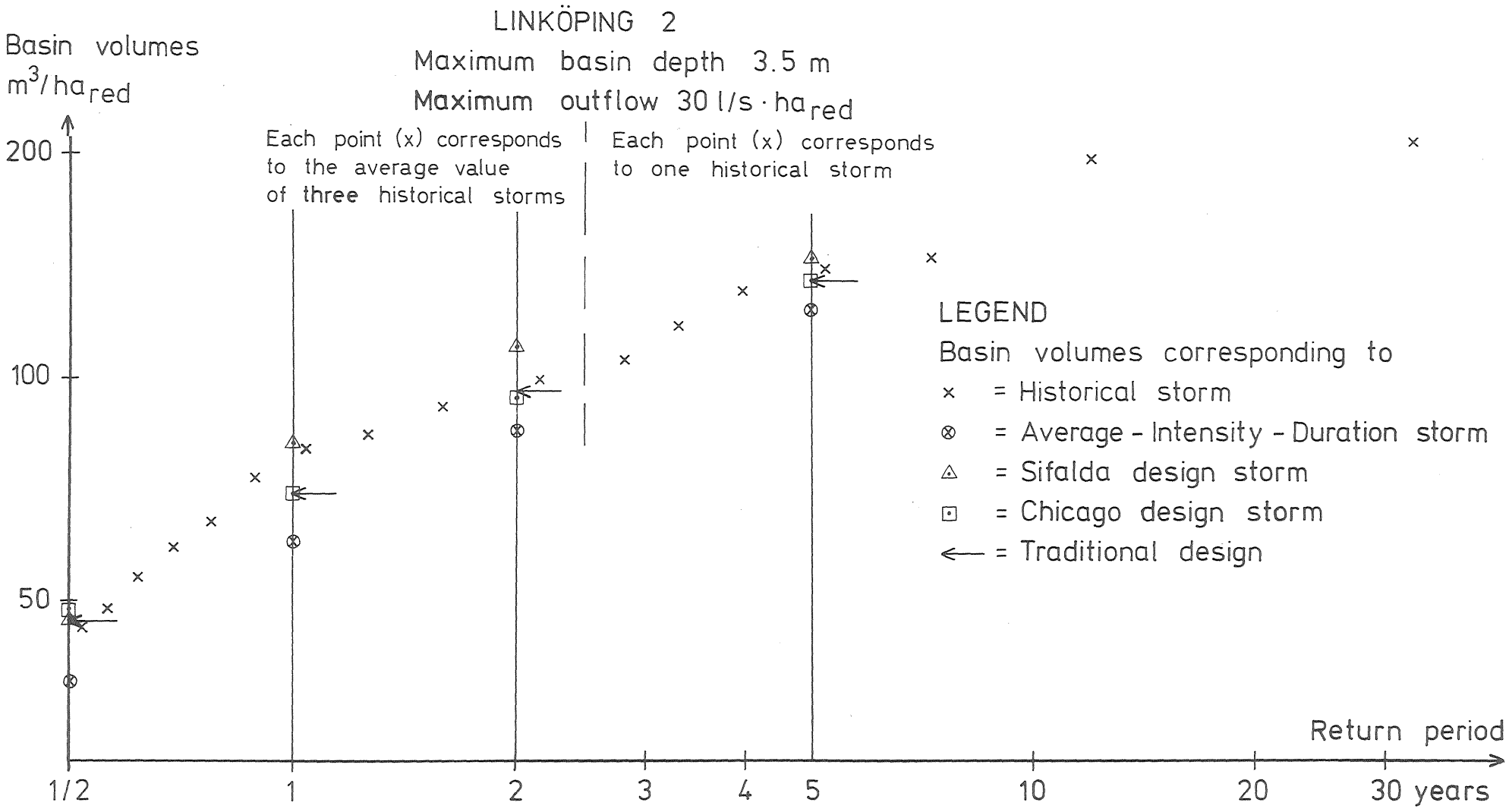

LINKÖPING 1
Maximum basin depth 3.5 m
Maximum outflow 20 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years


LINKÖPING 1
Maximum basin depth 3.5 m
Maximum outflow 30 l/s · ha red
Basin volumes
m3/ha red
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
100
50
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years


LINKÖPING 2
Maximum basin depth 3.5 m
Maximum outflow 5 l/s · ha red
Basin volumes
m3/ha red
500
400
300
200
100
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years


LINKÖPING 2
Maximum basin depth 3.5 m
Maximum outflow 10 l/s · ha red
Basin volumes
m3/ha red
400
300
200
100
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years


LINKÖPING 2
Maximum basin depth 3.5 m
Maximum outflow 20 l/s · ha red
Basin volumes
m3/ha red
300
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years


LINKÖPING 2
Maximum basin depth 3.5 m
Maximum outflow 30 l/s · ha red
Basin volumes
m³/ha red
200
100
50
Each point (x) corresponds to the average value of three historical storms
Each point (x) corresponds to one historical storm
LEGEND
Basin volumes corresponding to
× = Historical storm
⊗ = Average - Intensity - Duration storm
△ = Sifalda design storm
⊡ = Chicago design storm
← = Traditional design
Return period
1/2
1
2
3
4
5
10
20
30 years

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