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HYDROLOGIC SIMULATION METHODS IN RESERVOIR PLANNING

by

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1. Introduction

Hydraulic engineers were always involved in complex planning problems which required to match the demand for water over time and space by redistributing the supply of water. The technical means by which this was done is known since time immemorial. It is the reservoir with controlled release. But it has been only recently that they learned to solve their planning problems by analytical methods: the methods of systems analysis. System analysis has been accepted for reservoir planning by the academic water resources planner with great enthusiasm, and over the years the water resources literature has become a depository of sophisticated systems theoretical developments which in the eyes of the practicioneer serve more the purpose of solving intricate mathematical problems than real world tasks. It is indeed difficult for anyone to understand or even master all the methods and techniques for analysing water resources systems that are proposed, and often the frustrated student of the literature finds after he has finally developed the background to understand the papers of last year, that the problems that were treated had little to do with the problem which he wants to solve, while the present trend in theory involves new methods, mathematics and terminology which he does not understand. In such a situation, which is typical of a newly evolving field, it is extremely difficult to determine "the state of the art", which must be the basis of engineering design, so that the engineers tend to stick to the old rules until the new ideas have been tested and found to yield better results and better codes of practice than the old ones.

The transfer of knowledge from the realm of science to the real world of the engineer is impaired not so much by the unwillingness of the engineer to accept new and better methods, but by the failure of the scientist to communicate in terms of a language which the engineer can understand without needing to spend an excessive amount of time. The engineer usually works under restrictions in time and money, and he has to worry about many other problems which to him are of equal importance. On the other hand, it very often is overlooked by the engineer what help he can get from the scientist, who in turn is unaware of the need for help because the engineer does not communicate it to him.

It is the role of the engineering educator and academic consultant to bridge the gap between science and engineering: by filtering out from the theoretical work that which is not of immediate use, by applying the useful methods to actual problems in an exploratory manner through class room exercises and thesis work, thus finding out advantages and limitations, and by feeding back to the scientific community those problems that need theoretical treatment. By assuming this role, the writers have been able to help develop a number of techniques which have been found useful in the design of reservoirs, and which will be described in this paper. The discussion shall concentrate on single-purpose, single-unit reservoir systems.

2. Reservoir design as a simulation problem

In view of the general remarks made before, it seemed important to solve such reservoir problems by methods which can very easily be understood by engineers, and where every step in the development is transparent and easily checked. The method which fits these criteria is the method of simulation. For this method, the reservoir is presented by the continuity equation:

$$s_{i+1} = s_i + q_{i+1} - r_{i+1} - \ell_{i+1} \quad (1)$$

where the input function q_i denotes the amount of water entering, r_i the amount of water release, l_i the losses of water through evaporation and infiltration, and s_i the amount of water stored at time i during the unit time interval. All units are in m^3. The release r_i consits of two parts, as is indicated formally in Fig. 1. The first part is the release r_{1i} which is controlled by the demand d_i set by the decision maker. The second part r_{2i} is uncontrolled, either because there is an opening in the reservoir without controllable gates, or because a spill occurs over the spillway.

It is the purpose of the reservoir design problem to determine the size of the reservoir s_{max} and the operating rule for the reservoir. The operating rule:

$$r_{1i} = f(\pi_j, \ldots\ldots) \qquad (2)$$

is that rule by which gates are opened or closed, and which depend on decision parameter π_i, which can be modified by the operator, and external quantities, such as the reservoir content s_i. The parameters s_{max} and π_i are to be found for the reservoir problem in such a way, that the demand is met as well as possible. This condition is expressed through the objective function Z:

$$Z = \sum_i f_{2i}(d_i - r_{1i}, \ldots\ldots) \qquad (3)$$

which depends on the difference between demand and release, as wellas on other parameters, for example on seasonal weights. The vaguely expressed condition "as wellas possible" must be expressed in terms of a minimization or maximization of the objective function, i.e. one must find:

$$Z^* = \sum_i f_{2i}(d_i - r_{1i}^*, \ldots) \rightarrow \text{minimum (or maximum)} \qquad (4)$$

through a suitable choice of parameters $\pi_i = \pi_i^*$. Together with the constraints of the system, for example such as the maximum possible reservoir size or the maximum available water for a possible release, Eqs. 1 to 4 form the system analytical formulation of the single purpose, single unit reservoir problem.

It is easily possible, in principle, to solve this problem by means of simulation.

Fig. 2 shows the principle. The input into the system is given by q_i which can either be obtained through measurement, or by means of artificially generated data. The system is the reservoir whose equation is the continuity equation which can be interpreted as a transformation equation from the state s_i to the state s_{i+1}. The decision variable is the release r_{1i} which can be modified systematically by varying the parameters π_i over the whole range of their possible values. For each set of parameters, the value of the objective function is determined by running all the input data through the reservoir model and determining for each release the value of the difference $r_{1i} - d_i$, and then for all the input data the value of Z. The set of, say, k parameters and the corresponding value of Z form a k+1 vector which is stored in the computer. When all parameters have been varied over all their values – for example, if each parameter is assumed to have m possible numerical values, then m·k simulation runs must have been made – then the optimum value of Z can be found by direct search of the set of all vectors. The corresponding set of parameters π_i^+ must then be used for design.

The simulation scheme which was described above has great conceptual advantages over other types of operations research models. It runs exactly as the actual process does, and all the steps which are necessary to obtain solutions to the design problem are almost self evident and can be explained very easily. Also, if there is a good reason to modify some of the parameters, for example for the purpose of a sensitivity analysis, in which the effect of possible parameter errors are evaluated, this can also be done readily, and thus makes it a very flexible method. Detrimental is the use of direct search when there are too many parameter variations to consider, or if the generated data input is too long. Other operations research methods may then become indicated in order to reduce the number of simulation runs which is required.

In the following chapters, the simulation scheme of Figs. 1 and 2 and Eqs. 1 to 4 will be applied to some typical examples. They are taken from practical experiences of the writers and consist of a water supply problem and a flood protection research project in West Germany, and of a water supply reservoir project for irrigation in an arid country. It is most important to point out at the outset that although all three problems are solved by the simulation scheme outlined above they require entirely different considerations. These range from the large difference in the unit of time, - which for the water supply reservoir is one month, for the flood protection reservoir is one hour, and for the irrigation reservoir is one day, - to the entirely different demand functions, - which for the supply reservoir is a periodic function with period of one year, for the flood protection reservoir is an adaptive function of minimum flood damage, and for the irrigation reservoir is a forecasted water demand of irrigable crops. It may be concluded from these examples and from our experience with similar cases that it is not useful to try to develop a model which can be applied universally to any simulation problem involving single-purpose, single unit reservoir systems, but instead one should make available building block type sub-models which for each problem are selected to suit the special requirements, and which are assembled into a problem - specific composite model. In this manner not only single purpose, single unit reservoir problems can be solved, but also problems involving large catchments with many different units. It has been shown that most models for the water resources of large river basins which have been reported in the literature can be decomposed into a number of well known building blocks, which can be used in many different composite models (Cembrowicz et al., 1978).

3. Design of a water supply reservoir

The classical method of designing a water supply reservoir is obtained by using Eq. 1 with a release rule given to r_{1i} = constant for all i, and by using a historical sequence of discharge data as input q_i. The problem that is usually solved is to find that size s_{max} of the reservoir which is required so that the reservoir never runs dry. The graphical method by which this problem is solved is the well known Rippl - diagram (Chow, 1964).

There are several reasons why instead of wing this method one should work with artificial data. Thie first is the problem of the historical sequence: because the historical sequence is of a duration determined alone by the time of measurement T_D, the resulting reservoir size also becomes a function of T_D. It is found that the longer T_D, the larger, on the average, will s_{max} become. Thus, the longer we measure, the larger will be our reservoir. As a consequence, the first thing to do is to specify a duration, for example the design life of the reservoir, and to use a sequence of data of length equal to this duration. Only in exeptional circumstances is this amound of data available, and it is necessary to obtain additional artificially generated, data, and to use them in simulation for the design of storage reservoirs for water supply (Schultz, 1973).

The second reason is dictated by the fact that the maximum reservoir size s_{max} determined in the above manner is a random variable: for each sequence of artificial or historical data of the same duration T_D we can expect to obtain a different value of s_{max}. It is therefore necessary to generate a large number of input time series so that the statistical properties of s_{max} can be determined and used for design purposes. For example, from the probability distribution of s_{max} that value of s_{max} might be determined which in x out of n cases suffices to fully satisfy the demand rule, or which in at most x out of n months runs dry, on the average.

The third reason for using simulation by means of artificial data is that the simulation model permits to use complicated and even time varying operation rules. For example, in this manner the gradual decrease of the storage caused by silting can be accounted for, as shall be described in section 5.

For these reasons we have used simulation methods with artificial data as a routine method for reservoir design in many different water supply applications. An example shall be given in this section, which apart from its general applicability might also be interesting because of the important modification of the Thomas-Fiering model which was employed.
The problem consisted of finding the probability of failure of a reservoir of

maximum storage capacity s_{max}, whose stage-area and stage-volume curves are shown in Fig. 3. The input into the reservoir is specified through measured discharges, which have been recorded once daily over a period of 27 years. The release rule is dictated by the water demand of a nearby city, which can be described by a seasonal demand rule with mostly periodic components, but with a certain amount of on-line adaptation to exceptional water demand during very hot seasons.

3.1 Defining the probability of failure

There are many ways of defining a probability of failure, among them the probability of the reservoir failing in any one year, which is given to

$$PY = N'/N$$

where N′ is the number of years during which the reservoir could not meet the demand and N is the total number of years of input data. Another definition is given by the ratio of the number of months during which the reservoir fails in the total number of months of the input data. This is denoted by PM. Other definitions are also conceivable, such as the ratio of the amount of water which could not be delivered to the total amount of water which was demanded, and other similar rules. The problem considered has PY and PM as design criteria.

3.2 The input rule

The measured discharge data have been used to determine the parameters for a model for generating artifical data. The model is a generalization of that of Thomas and Fiering (Maass et al., 1962).

$$q_{m,i+1} = \bar{q}_{m,i+1} + \rho_{i+1,i}\left(q_{m,i} - \bar{q}_{m,i}\right)\frac{\sigma_{i+1}}{\sigma_i} + t_{m,i+1}\,\sigma_{i+1}\sqrt{1-\rho^2_{i+1,i}} \quad (5)$$

Here $q_{m,i}$ is the volume of water flowing into the reservoir during month i and year m, and $\bar{q}_{m,i}$ is the average value of $q_{m,i}$ for the year m which is generated from the long term monthly mean value q_i for the month i through the relation:

$$\bar{q}_{m,i} = f_m \cdot \bar{q}_i \qquad (6)$$

where f_m is a factor embodying the variability of the annual mean value, as will be discussed below. The correlation coefficients $\rho_{i+1,i}$ are those of adjacent months i and i+1 and are defined in the usual manner. Similarily, σ_{i+1} and σ_i are the standard deviations of the monthly values $q_{m,i}$ and are equal to the standard deviations of the measured data series.

The quantity $t_{m,i+1}$ is a random variable with mean O and standard deviation 1. In the original Thomas-Fiering model, this quantity is distributed according to a normal or gamma distribution. We have tried many different distributions and find that the Weibull distribution, whose probability distribution is described by:

$$F(t_m, i) = 1 - e^{-\lambda_i (t_{m,i} + \mu_{ti})^{\frac{1}{\alpha_i}}} \qquad (7)$$

fits the data as good as any other distribution. It has the advantage that its parameters are easily found from the relations:

$$\left(\frac{\sigma_{ti}}{\mu_{ti}}\right)^2 = \frac{2\Gamma(2\alpha_i)}{\alpha_i \Gamma^2(\alpha_i)} - 1 \qquad (8)$$

$$\lambda_i = \left[\frac{\Gamma(\alpha_i + 1)}{\mu_{ti}}\right]^{1/\alpha_i} \qquad (9)$$

and

$$0 = \bar{q}_{i+1} - \rho_{i+1,i} \frac{\sigma_{i+1}}{\sigma_i} \bar{q}_i + \mu_{ti+1} \sigma_{i+1} \sqrt{1 - \rho^2_{i+1,i}} \qquad (10)$$

Here, Eq. 10 is a condition which arises from the requirement that $q_{m,i}$ and $q_{m,i+1}$ can never be smaller than O. It is obtained by inserting

$q_{m,i} = q_{m,i+1} = 0$ and $\bar{q}_{m,i} = \bar{q}_i$ as well as $\bar{q}_{m,i+1} = \bar{q}_{i+1}$ into Eq. 5 and solving for $t_{m,i+1} = \mu_{t_{i+1}}$. Eqs. 8 and 9 arise from the usual defintions of mean value and standard deviation. They are solved for α_i and λ_i by using tables for the gamma distribution, which are part of the program package for most computers. They must be determined for the distribution of each month from the experimental data record. For the data of the present case they are listed in table 1.

The main advantage of using the Weibull distribution Eq. 7 is that it can be solved explicitely for $t_{m,i}$ through the relation:

$$t_{m,i} = \left[-\frac{1}{\lambda_i} \ln \left\{ 1 - F(t_{m,i}) \right\} \right]^{\alpha_i} - \mu_{t_i} \tag{11}$$

This is very convenient for the procedure of generating artificial data. Since *any variable x with probability distribution* F(x) *can be transformed into the* Weibull variable t through the relation F(x) = F(t), it is easy to generate random data t from a set of uniformly distributed data between zero and 1 for which y = F(x).

The original Thomas-Fiering model (Maass et al, 1962) differs from Eq. 5 through the condition $q_{m,i} = q_i$, that is, the original model has a monthly average inflow which was considered the same for all years. It has the disadvantage that it does not preserve the autocorrelation of the annual mean values. In fact, if a model is used based on the original Thomas-Fiering model, one finds that the annual mean values are quite uncorrelated, the correlation coefficient r_1 between adjacent years usually fall below 0.1. But it has been known since the important investigations of Hurst (1951) and Yevjevich (1967) on the probability distribution of the range, that in order to yield a reasonable estimate of the rate of failure of a reservoir, the generation model for the discharges must preserve the "Hurst coefficient" h. Hurst found the following relation:

$$\frac{R}{\sigma} \sim n^h \tag{12}$$

where R is the range, σ is the standard deviation, n the length of the serie and h the Hurst coefficient with $h \approx 0.7$.

The models which have been reproduced to preserve the Hurst coefficient, such as the "broken line process" (see Garcia et al., 1972) or the "fractional Gaussian process" of Mandelbrot and Wallis (1969) have the advantage of preserving the value of h, but are found to give severe difficulties in the actual application, quite apart from not having enough flexibility to preserve the annual cycle of the parameters of the distributions of monthly discharges or flow volumes. The better solution in our opinion is to modify the Thomas-Fiering model to make it preserve long range persistence, and it is this reasoning that led to the introduction of the factor f_m mentioned above (Treiber, 1980, to be published). According to this modified Thomas-Fiering model the annual mean values are described by a Markov-1 model:

$$\bar{q}_{m+1} = \bar{q} + \rho_1(\bar{q}_m - \bar{q}) + t_{m+1}\sigma_{\bar{q}}\sqrt{1-\rho_1^2} \qquad (13)$$

where ρ_1 is found from the experimental data record, σ_m = standard deviation of the annual mean values $\bar{q}_m$, and t_{m+1} is a Weibull distributed random variable whose parameters are found from applications of Eqs. 8 to 10 to the sample of mean annual data. With this model, the autocorrelation for the 27 years record can be approximated, but one must realize that exact equality is not required because of the sample variablility of h and the short duration of the generated data series.

The generation procedure then is modified as compared to that for the Thomas-Fiering model by first generating a sequence of mean annual values q_m for each year m, by means of which the factor f_m is calculated to:

$$f_m = \bar{q}_m/\bar{q} \qquad (14)$$

where $\bar{q}$ is the long term mean value, i.e. the mean value of the data record.

Typical results for generated data are shown in Figs. 4 to 6 for which the Weibull parameters of the sample record table 1 are used. It is seen that a

month i	α_i	λ_i	μ_i
1	0.813	0.926	1.172
2	0.287	0.548	1.762
3	0.487	0.671	1.465
4	0.308	0.562	1.723
5	0.944	0.961	1.040
6	0.694	0.798	1.245
7	0.950	0.965	1.036
8	0.714	0.810	1.227
9	0.331	0.576	1.634
10	0.899	0.930	1.074
11	0.108	0.418	2.246
12	1.020	1.014	0.986

Table 1: Parameters of WEIBULL-distribution

sequence of 100 records of 60 years duration which is patterned after an observed stream flow record of 27 years preserves well the parameters of the monthly moments. Also, the observed lag-one correlation coefficient ρ_1 of 0.298 is in exellent agreement with the correlation coefficient of 0.296 calculated from the data records.

3.3 The release rule

The release rule which was used required that $r_{1i} = d_i$, where r_{1i} is the controlled release during month i, and d_i is the demand. If the demand cannot be met, then the corresponding month is one where the reservoir failed. If the release has to be larger than d_i during month i in order to prevent the reservoir from overtopping, then the reservoir also fails in some other sense, because the uncontrolled discharge cannot be utilized for the water supply purpose. This latter effect is reflected in the ratio $\mathbb{R}$ defined to

$$\mathbb{R} = \frac{\sum_i r_{2i}}{\sum_i q_{m,i}} \quad i = 1, 2, \ldots 720 \qquad (15)$$

In principle, the demand function also is a random variable; however, for the present problem it was sought to utilize the water of the reservoir as a sort of base load and to supply the missing water for the water supply of the whole city from other sources. Thus, a demand rule was used according to which 60 % of the average available annual water inflow $\bar{q}$ was used during the summer months, while 40 % was used for the winter months. In addition, an increased release was required during exeptionally hot summer months, which could occur in July or August. The historical record showed that 13 out of a total of 54 months of July and August were extremely hot, thus for each of these two months whenever it occured during the generation scheme a random number between 0 and 1 was drawn. If it was below 13/54, the month was considered an extrem month with an increase in demand by 40 %. If it was above the ratio 13/54, the month was considered ordinary. As a final condition on the demand it was required that the release be reduced whenever the content of the reservoir fell below a minimum of 14 million m^3.

3.4 Results of the simulation runs

Typical results obtained with the rules described in sections 3.1 to 3.3 are illustrated in Figs. 7a and 7b, and in table 2. Fig. 7a shows the results obtained with the Thomas-Fiering model for a typical sample record of 60 years. Fig. 7b is for the modified model. The top curve is a trace of the generated inflows, noticeable is the persistence of the dry months around month 450 for the modified Thomas-Fiering model, with the resulting emptying of the reservoir, which persists for a prolonged time of almost 40 months. Such dry spells cannot occur with a reasonable probability for a run according to the Fiering model, while through the effect of ρ_1 it is not uncommon to occur for the modified model. This is the reason for the different probabilities of failure for the two cases which are listed in table 2.

The other two traces in Figs. 7a and 7b are the releases and the reservoir content, respectively.

$\bar{q}$ [m^3/a]	ϱ_1	$\mathbb{R}$	PM	PY	Model
$33.43 \cdot 10^6$	-0.035	0.175	0.001	0.005	Fiering
$32.43 \cdot 10^6$	0.296	0.193	0.022	0.070	modified Fiering
$32.6 \cdot 10^6$	0.298	0.200	0.000	0.000	original data

Table 2: Results of simulation runs

4. Reservoir operation for a flood protection reservoir

In the densily populated Federal Republic of Germany industrialization and housing developments have moved the urbanized areas nearer to the river and up the valleys, so that floods in these rivers cause more damage than they used to do, in particular since urbanization tends to increase flood peaks (Wittenberg, 1975). The engineering solution to the flood problem has been more and more to build flood retention reservoirs designed to keep down the flood damage. However, very often the size of the available reservoir site is not sufficient to retain, for example, the hundred year flood, and small reservoirs come into existence which may not be very efficient unless the releases from them are controlled in such a way that they are fully effective in controlling the outflow according to the desired objective function. Objective functions which are based on financial considerations are usually not very well defined for flood protection reservoirs, it seems that better objective functions are based on rules on the desired modification of the flood to be accomplished by the reservoir operation. How these could look is indicated in Fig. 8 (from Plate and Schultz, 1972), according to which an operation rule for the reservoir either is designed to yield minimum flows downstream of the reservoir, or to yield a flood of the shortest duration. The former rule is the usual one for protecting riverside property, but the latter may also be desireable in some cases, for example if there are agricultural crops in the flood plain, which do not suffer so much from the flooding as from the length of time during which the crop is under water. For both cases it is evident that the release from the reservoir cannot be decided without advance knowledge of the flood to come, i.e. a method is required to forecast the flood wave.

From this brief description it is seen that the operation of a flood protection reservoir of limited storage capacity involves the solution of two important theoretical problems: the problem of real time forecasting of the flood, and the problem of optimum operation of the reservoir once the flood to come is known.

4.1 Forecasting of the flood

The problem of forecasting the flood wave is one of the most difficult problems in operational hydrology because of the large random components involved in the runoff process, and because of the fact that the reservoir is usually capable of protecting against smaller floods. The latter implies that a good forecast is required only for extreme cases, such as the floods whose recurrence interval is larger than, say, 30 years. For smaller floods a release is not critical because the reservoir will not be filled by the end of the event, so that a release according to some simple rule such as according to a constant value of $r_{1i} = r_{1d}$ is entirely adequate. It is clear that it is an expensive proposition to require all the equipment for on-line forecasting and control which is used only once in a generation. Thus, on-line operation based on forecasts of rainfall or runoff are useful only in such cases where either the goods or property to be protected is so valuable, or the protection accomplished through on-line operation is so useful, that the long waiting times can be tolerated. Or where protection of limited extent is desired, for example in sewage disposal systems, where it is useful to protect against floods of high recurrence, while accepting the damages due to the occasional extreme flood.

The other difficulty of on-line forecasting is caused by the randomness of the rainfall - runoff process which depends on many factors which are time varying in a manner which is almost impossible to predict. Forecasting thus involves considerable uncertainties, which put in question the usefulness of the forecast and the control of the reservoir which is based on it. For if during the rare extreme flood event a forecast is made which yields too large a flood wave then it is conceivable that the release is increased to a level which causes damage, while the reservoir remains part empty until the end of the flood - one must imagine the consequences if this becomes known to the people which suffered damages! The opposite occurs when the forecasted flood wave is too small: the reservoir is filled up too early, and perhaps when the flood peak comes it must be released unchecked over the spillway of the dam.

A problem-specific forecasting method for flood waves has been developed by Anderl (1975). It is based on the theory of the unitgraph, according to which the flood wave can be predicted from the known effective rainfall distribution I_i by means of a convolution with the IUH (instantaneous unit hydrograph) h_i where h_i is the event-specific unit impulse response of the best fitting linear system equivalent of the catchment, so that:

$$q_i = \sum_{j=1}^{i} I_j h_{i-j+1} \cdot \Delta t \qquad (15)$$

If the system function were a deterministic function and no deviations could occur, then the forecasting problem would be reduced to a forecast of I_i, which requires a forecast of both losses and actual rainfall. The methods which can be employed for this forecasting are at present being developed: it is anticipated that from past rainfall records and satellite pictures it will eventually be possible to forecast future effective rainfalls for large storms. However, some good forecasts of floods, which are suitable for developing effective control strategies for short times, can already be obtained by using the rainfall as input, which has fallen to the present.

Due to non-linearities and random errors, however, h_i is not a deterministic function but changes from event to event. The method of Anderl (1975) was designed to minimize the effect of this randomness. It calculates for each forecasting interval a correction to the mean IUH h_i which is denoted by h_i' and which is determined as that correction which minimizes the error between measured and calculated runoff. The method is based on the assumption that the effective rainfall is known for the past $i \leq p$, that the true discharge q_i is available up to time $p \cdot \Delta t$, and that it has been possible to calculate from previous floods the mean IUH $\bar{h}_i$. Then a calculated discharge q_{ci} can be found for $t \leq p.\Delta t$:

$$q_{ci} = \sum_{j=1}^{i} I_j \bar{h}_{i-j+1} \cdot \Delta t \qquad (16)$$

and by subtracting the calculated discharge from the true value, an error discharge Δq_i can be found, which is sketched schematically into Fig. 9 This error discharge is continued into the future up to a time $p \cdot \Delta t + 1_v$, which is that time during which runoff can be expected from the rainfall which has fallen up to time $p \cdot \Delta t$. The continuation is performed by fitting an exponential function to the function q_i which over the time $t = 1_v$ decreases from Δq_p to $\Delta q_e = 0.01 \Delta q_p$. Thus one obtains:

$$\Delta q_{pi} = K^{i-p} \Delta q_p \quad \text{for } i > p \quad \text{with} \quad K = 0.01^{\frac{\Delta t}{t_v}} \qquad (17)$$

By means of the continuation of Δq_i it is possible to solve the set of equations for h_i' by the method of least squares, in the same manner as solving the original data of the past records for the mean value $\bar{h}_i$ of the IUH. With these values of $\bar{h}_i$ and h_i', the forecast q_{pi} is obtained on the assumption that there will be no more rain after time $p.\Delta t$:

$$q_{pi} = \sum_{j=1}^{i} I_j h_{i-j+1} \cdot \Delta t \qquad i > p \qquad (18)$$

The accuracy of such a forecast can be determined by calculating a dimensionless mean-square error:

$$\sigma_c = \sqrt{\frac{\Sigma (q_i - q_{pi})^2}{n}} \cdot \frac{100}{q_{max}} \qquad (19)$$

With this goodness-of-fit criterium some forecast results are shown in Fig. 10. The main results of this figure is that a forecast with an error of less than

5 % is obtained only if the flood wave has developed past the inflection point - a result which is confirmed by numerous other examples. On the other hand, if the forecast has not to extend over the whole duration of the flood event, then better agreement can be found, with accuracies which are quite comparable to those obtained by more refined methods, such as the use of the Kalman-Bucy filter (Hino, 1973; Wood, 1978) for the forecast of the unit hydrograph of future runoffs.

In conclusion it should be pointed out that the forecasting procedure used here is only part of the necessary calculations. The adaptation of the losses (i.e. the calculation of an event-specific effective rainfall) is of equal or larger importance, but it goes beyond the scope of this short survey to go into methods for the forecasting of losses, exept for stating that the methods are similarly structured as the one described, because it is possible to formulate the loss function in a similar manner as the rainfall-runoff hydrograph.

4.2 Optimum control for known input functions

Once the input into the reservoir is known from the forecast it is easy to calculate the optimum release. Let us briefly consider the operation of the flood protection reservoir for the objective of minimizing the maximum discharge $r_i = r_{max}$. It can be shown for example by means of the method of Dynamic Programming, that among all possible curves f(t) with $f(t) > 0$ for all t, and with a constant area A over a constant base length T, the curve $f(t) = c = \text{constant} = r_{max}$ is the one with the minimum peak value. Thus, it is desireable to operate the reservoir in such a fashion that the release from the reservoir over the time of maximum flood level is kept constant and as low as possible.

There are two methods which we have used to solve this problem of finding the minimum value of r_{max}. The first involves simulation. Starting with a large value of r_{max}, the reservoir content was determined for the flood wave using the continuity equation, Eq. 1. If s_{max} calculated by this method was smaller than the actual available reservoir storage, then r_{max} was gradually decreased until calculated and actually available storage were equal. This method is easy to apply and uncomplicated, but it does not work if more than one reservoir is operated simultaneously, and the purpose of the system of reservoirs is to control flood damage at a point downstream of all reservoirs. For such a system, it is possible to use dynamic programming (Plate and Schultz, 1972). Examples of reservoir operation determined in this manner have been described by Schultz and Plate, (1976), Mayer-Zurwelle (1973).

4.3 Further developments

As has been described above, it is very unlikely that the implementation of an adaptive control system for the purpose of flood control for rare floods ever becomes economically feasible, unless it is possible to set up an extreme flood prediction center which serves a very large area, in which extreme events occur somewhere a few times during a year. In West Germany, it is not very likely that the money which is required for setting up such a center becomes available very soon. It is therefore very useful to ask the question of how an automatically controlled system of reservoirs, which have either fixed size openings or gates which can be controlled automatically, must be operated so that the operation of the system approximates most closely that of the optimum system with adaptive control.

Another problem is the answering of the question of which of the reservoirs in a system of many must be constructed first in order to minimize expected damage due to floods over the life of the reservoir system. This problem has been approached by Bogardi (1979), who used a single extreme flood as indicator of the performance of the system and optimized the sequence of the reservoir construction by a branch and bound algorithm. The system was severely constrained by the fact dictated by fiscal considerations that only one reservoir at a time could be built, that construction of a new reservoir could only be started once every year, and that the amount of money available for construction was constrained. For the details of this program, reference is made to the work of Bogardi (1979). Recently we have amplified this problem by including not only one flood wave into the model, but by using simulation with *artificially generated* multi site runoffs determined from a multi site rainfall generation model (Binark, 1979) as input into local unit hydrographs for each reservoir. It is not possible to optimize such a model, nevertheless, it was possible to check the performance of some simple operation rules (for example of keeping the reservoir fully open until a certain flood level is reached, then close it completely until the end of the flood wave), and to find an optimum sequence of constructing the reservoirs.

5. Reservoir design for irrigation in an arid country

Irrigation in arid countries usually is limited by the water which is available for irrigation, and the problem is to find that area which can be irrigated with the available water. We have investigated such a problem for an area located in the Southern part of Saudi Arabia on the coast of the Red Sea. Here, the available water is largely runoff which results from strong convective, seabreeze induced precipitation in the foothills of the Azir mountains, which runs off as flash floods towards the sea in wadis which carry usually no or very little discharge, but which during exeptionally heavy rainfalls may grow into streams with discharges of hundreds to thousands of m^3/s. Because of the barren structure of the land, the flash floods carry very large amounts of sediment, so that a system for improving irrigation of the area requires a reservoir which has the double function of breaking the flood wave and of storing the water for redistribution. The time of storage must be held as short as possible in order to prevent exessive losses due to evaporation, but it must be long enough to supply water for the crops during the time of need. Such a reservoir in which water is retained for some time will have a high trap efficiency for the sediment and it is necessary to have a reservoir size which is large enough to function properly for a long enough time before it is silted up to uselessness.

The problem which had to be solved was to find the area which should be developed for irrigation. It was not possible to give an objective criterium for a design decision, so it was assumed that a good design is to develop that area which in 80 % of all years is large enough for the available water, which means that in almost 80 % of all years the area cannot be irrigated completely, while in 20 % of all years some of the water had to be carried over from one year to the next. Losses of water occured through evaporation and spill due to overflow of the reservoir whose size was limited by the storage volume which could be accommodated at the best suited site.

5.1 The simulation model

The best method for solving this problem is simulation. Due to the special situation it was found best to write the continuity equation Eq. 1 for daily time steps. The time unit of one day permitted to account for the flash flood character of the discharges, and also for the sediment transport, for which two additional equations are required. The first is a rating curve, by means of which a relationship between water inflow q_i and corresponding sediment load q_{si} is established:

$$q_{si} = A q_i^{\beta} \tag{20}$$

There exists at present no reliable method by which the coefficients of Eq. 20 can be found from first principles, and it is customary to find these coeei-cients empirically from local measurements. Typical exponents β lie between 1.5 and 3 (Garde and Ranga-Raju, 1977) while the coefficient A depends on catchment properties as well as on the characteristics of the rainfall. Some experiments conducted at the future site of the dam yielded a function for the total sediment load q_{si}:

$$q_{si} = 14000\, q_i^{1.5} \tag{21}$$

where q_{si} is in to and q_i in 10^6 m^3, both for daily values.
The second equation describes the continuity of the volume of the reservoir:

$$s_{max\, i+1} = s_{max\, i} - \varphi_T\, q_{s\, i+1} / \gamma_s \tag{22}$$

where φ_T is the trap efficiency of the reservoir, and γ_s the specific weight of the deposited sediment (in to/m^3). By means of considerations which go beyond the scope of this paper the trap efficiency was determined to 0.8, while the specific density was taken from experience with reservoirs in the same area to be 1.4 to/m^3.

The third equation to be used is the continuity equation for the water, Eq. 1. For the present case, a double index is used, with i describing the time, and j the season. This is necessary because the loss due to evaporation depends on the climatological variables temperature and average humidity, as well as on the radiation budget, as described by the well known method of Penman (1948). By this method, monthly evaporation rated Ej for the j-th month were calculated on the basis of long term averages of the climatological data, where Ej is the height of the column of evaporated water in m/month, from which for a month with n_j days the daily loss l_{ji} is found to be

$$l_{ji} = \frac{E_j}{n_j} A_{ri} \tag{23}$$

where A_{ri} is the surface area of the reservoir for the volume s_i on day i, in m^2, which is found from the usual storage-height and area-height relations for the reservoir in question.

With this expression one can formulate the continuity equation to read:

$$s^o_{ji+1} = s_{ji} + q_{ji+1} - r_{ji+1} - \ell_{ji+1} \tag{24}$$

where s^o_{ji} is an auxiliary variable which is determined by the constraints of the problem:

$$s^o_{ji} = s_{ji} \qquad \text{if } s^o_{ji} \leq s_{max\,i} \tag{25}$$

$$s^o_{ji} = s_{max\,i} \quad \text{if } s^o_{ji} > s_{max\,i} \text{ and } s^o_{ji} - s_{max\,i} = r_{2i} \tag{26}$$

where r_{2i} is the spill. In order to solve the set of Eqs. 23 to 26 it is necessary to know the inflow function q_{ji} and the release function r_{1ji}, where the latter depends on the demand function d_{ji}.

5.2 The inflow model

The inflow function must be a time series of daily flows which has all the characteristics of the historical time series observed at a gage. The observed time series was very short, and it represents a very incomplete sample of the ensemble of possible inflows. It was therefore decided to generate artificial time series of inflows, each of a duration of 100 years, and each with the statistical characteristics of the historical short duration sample. The method of data generation employed was based on a model developed by Treiber (Plate and Treiber, 1979) which had been adapted to the particular data and flow characteristics of flash flood type inflows (Schmidt and Treiber, 1980). This model is not elementary, because it is based on a combination of two different processes from which the inflow at day i is determined to:

$$q_{ji} = \delta_i \, t_{ji} \tag{27}$$

where δ_i is the result of an on-off process by means of which it is decided whether the day is wet ($\delta_i = 1$) or dry ($\delta_i = 0$), and t_{ji} is the result of a Marcov - 1 process with parameters which vary from month to month:

$$t_{ji} = \rho_j \, t_{ji-1} + z_{ji} \tag{28}$$

From Eq. 28, with z_{ji} a random, Weibull-distributed number, a continuous record of data is generated, which becomes intermittent through the factor δ_i which is found from the original data on the basis of transitional prohabilities.

Results of this generation procedure are shown in Fig. 11-15. From Fig. 11 a comparison can be seen of the original data record with a sample of the artificially generated data, which indicate the visually satisfactory agreement of the two data sets. This agreement is further exemplified by comparing statistics, such as the monthly mean $\bar{q}_m$ of the inflows, the standard deviation σ_m of the monthly mean, as well as similar characteristics for the daily statistics (with index i) indicated in Fig. 12-15. All generated parameters are seen to compare favorably with the statistics of the original record, which permits one to have considerable confidence in the validity of the model for matching the observed data record. What the method cannot supply, of course, is an improvement in the sample uncertainty: the short length of the original record makes this uncertainty very large, and it is necessary to continue oberservations of the flow data and to improve the parameter of the model until the very last possible moment before a final decision must be made on the structures or the size of the area to be developed.

5.3 The release model

The release function r_{1ji} from the reservoir is the decision variable, which must be oriented towards satisfying the demand d_{ji}. In reality, the demand is a random variable which depends on the growth history of the crops which are to be irrigated and on climatological factors, as well as on biological conditions, soil moisture conditions, and on the conditions of fertilization. Of these the most important variable which directly affects the need for irrigation water is the soil moisture content, and more sophisticated models (Norero, 1972), Minhas, 1974) of demand functions are based on this quantity. Here, only a simple demand function will be given, which is based on the assumption that enough water will be applied to all irrigated areas to replenish the soil moisture to field capacity. In that case, the water demand can be expressed by the relation:

$$d_{ji} = \frac{E_{cj}}{n_j} \cdot \frac{0.1 \, A_{ci}}{\varphi_c} \quad (m^3/day) \tag{29}$$

where A_{ci} is the area to be irrigated during day i, φ_c is the irrigation efficiency, which was taken as constant and equal to 0.35, and E_{cj} is the plant evapotranspiration during the month j. Again for simplicity sake a single crop is considered which is grain sorghum, for which experimentally observed water requirement coefficients K_i are used to calculate the water requirement of the four sorghum crops listed in table 3 . Thus, the water demand is a function of fixed parameters with the exception of A_{ci}, and it is the purpose of the irrigation model to optimize this area.

A crucial problem in the determination of the area A_{ci} is the amount of water which can be expected throughout the growing season. Because of the stochastic nature of the inflows, it cannot be expected that the same amount of inflow occurs during each year or month, thus one does not know at the

Table 3: Demand function of irrigated area

E_{cj} (in m^3/ha) for month j

j = month	crop 1	crop 2	crop 3	crop 4
Aug.	1370			
Sept.	2200			
Okt.	1740			
Nov.	100	1410		
Dez.		1960		
Jan.		630	890	
Febr.			1910	
März			440	1290
Apr.				1490

beginning of the irrigation season how much water will become available. But one must make a decision at the beginning of the irrigation season on how much of the available area should be planted, and this must be based on a forecast of the actual inflows. If this estimate is too large, one will find that there is not enough water in the reservoir at a later part of the growing season, and a part of the irrigated area will have to be abandoned without yielding a crop, in order to irrigate the reminder of the crop. If the estimate of the expected inflows is too low, water will be wasted, either by spills or by keeping it in the reservoir for the next growing season, thus subjecting it to too much evaporation. For the operation of such a reservoir it is therefore of great importance to find an adequate forecasting rule, and some effort has been spent on finding such a rule (Schmidt and Plate, 1980). At present, the best model that we have is based on a regression analysis of the total amount of water available during the growing season with the reservoir content at the beginning of the season, but some improvement can be expected from a more detailed analysis, which is at present underway. Another decision rule is based on the monthly inflows and on the reservoir content at the beginning of the season, in the form:

$$\sum_{j}^{h} r_j = s_j + S_j \qquad S_j = \sum_{j}^{h} q_{ji} \tag{30}$$

which has been used by Plate and Treiber (1979). In this equation, S_i is the total amount of water which is inflowing during the remainder of the growing season, i.e. from the beginning of month j, at which the decision of irrigating a certain area has to be made, to the time of the harvest h. It is a random variable, which is found to be gamma-distributed. A decision was made to use that value of S_i^+ for the design which was available in 80% of all cases. These values as found from the data are listed in table 4.

Results obtained with the decision rule for the area A_{ci} which was described above are shown in Fig. 16 . The calculations were based on the crop evaporation rates of table 3 the forecasting rule Eq. 30 with the forecasted values of table 4 and with the rule that the area A_{ci} was reduced to the area which could be fully irrigated whenever the available water was less than

water available from month k_1 (k_1 = \ k_2 =)	water available to month k_2: 1	2	3	4	5	6	7	8	9	10	11	12
1	2.2	4.2	6.2	6.9	7.6	7.9	8.1	8.5	9.8	12.3	13.4	17.2
2		1.0	2.9	3.6	4.3	4.5	4.6	5.1	6.2	8.8	9.7	13.9
3			1.2	1.8	2.5	2.6	2.8	3.2	4.1	6.4	7.2	11.3
4				.3	.8	.9	1.1	1.4	2.1	4.4	5.2	9.1
5					.3	.4	.5	.8	1.5	3.7	4.5	8.4
6						.0	.1	.2	.8	3.0	3.8	7.7
7							.0	.1	.7	2.8	3.6	7.6
8								.0	.5	2.6	3.4	7.4
9									.2	2.1	2.9	6.8
10										1.2	1.9	5.6
11											.1	3.2
12												2.4

Table 4: Sum of inflows from month k_1 to month k_2 during the growing season which can be met in 80 % of all years (in $10^6\ m^3$)

the required amount. The growing season for sorghum was assumed to extend over four possible crops, the second of which had the highest priority because it produced the highest yield per m^3 of available water, and the third and fourth of which were produced only if some water was left for them. The calculations were performed twice, once with no limitations on the area which could be irrigated with the results shown in Fig. 16, from which the design area was found to A_{cd} = 1500 ha, and once with a limit set by the design area A_{cd} on the maximum area which could be irrigated, with results shown in Fig. 17. The calculations were made by Hiessl (1979) from whose unpublished thesis the figures and tables of this section are taken.

6. Reservoirs as part of project planning

The simulation models which were described in the examples are all well posed problems with all the inputs and constraints known as far as they could be known. However, it is hardly ever possible to approach a reservoir planning problem in such a straight forward manner. Usually, the reservoir problem arises in the context of a much wider planning task, during which it is formulated and revised and reformulated many times. In particular, the single purpose single unit reservoir, although very frequently used, is hardly ever the centre of a project, which much more commonly consists of many units, and serves many different purposes. Usually it is formulated during a latter stage of the planning process for a project. The first stage consists of an ordering of the project alternatives on the basis of fairly broad information, and of the selection of that alternative which is best suited for the purpose at hand. In practice, this selection process is the "feasibility study". It can be approached by operations research methods (Haimes, 1977) but it usually is not, because the primary feature of the selection of the most desirable alternative is that it takes place in an intensive dialog of many different types of experts, of representatives of the customer, and, if all is planned well, of the beneficiary which is, in most cases of water resources projects, the people.

A general scheme of the planning process as it takes place, or should take place, in the planning of an irrigation project is shown in Fig. 18 . There are a total of six stages of the process which must be seen together, and within these the optimum choice of the reservoir is only one, and usually a comparatively small problem. The first stage is the response to the political decision to build a project involving irrigation - very often as part of a more general resources development project, which has to be formulated in a dialog among experts, administrators, and decision makers or politicians. The decision to do a project should then be followed by a program of collecting data on the basis of a very crude initial concept on how the project may look. This process of collecting data unfortunately is almost always not taken too seriously, usually the collection of data is part of a fixed bid feasibility study and if there is not enough time allowed for it, then the collection of data must suffer. The types of data that must be collected range from information on the construction materials available in the project area to sweeping evaluations of the sociological and economic changes which might result from the project. Of these data, the hydrological and agricultural data are especially important for the reservoir problem, but as a rule these data are usually very incomplete in a project area. It is therefore necessary that agricultural and hydrological field work be started as one of the very first sections of the project planning process, preferrably right after the tentative decision to build a project has been made. Agricultural field work should perhaps include the establishment of an Agricultural Research Station, while hydrological field work consists of the evaluation and upgrading of the existing network of raingages and discharge measuring stations, and the calibration of the latter.

It should be self understood, but it is not, that the data collection is not considered a purpose in itself, but is oriented toward the solution of the project planning problem. Very often it is found that the scientists who do for example the sociological or agricultural field work do not know what purpose their data serve in the general planning process. And here lies one of the advantages of a system concept such as the scheme of Fig. 18 : that it can serve as a framework for organizing the data collection process. If experts of every field know what is the information needed from them for the system concept to be applied, then data collection and evaluation can become very efficient. But this is easier said than done, because it is very difficult to communicate properly between experts of many different specialities. Each of them has been "brain washed' by the traditions of his field into believing that he knows exactly what should be done, and it is difficult for the system expert, who often knows very little about what is going on in the field to convince his colleagues that he should tell how the needed information must be prepared for his purposes.

The best way of overcoming the communication barriere is by letting every expert participate in the next stage of the planning process, stage 2,in which all data are entered into a system concept with the purpose of selecting the best overall solution among the alternatives which are available. It is of importance that this selection process must not be done by the water resources expert alone, but that it is to be carried out by all experts and has to be worked out in all its consequences and with due consideration to all constraints, where the water resources expert only formulates the problems and asks the questions on details which the other experts must answer. Sometimes the selection of the best alternative is done by operations research methods, by solving an optimization problem, but usually it is not. We find it very useful to determine by operations research techniques that solution which is most economical. This solution then serves as a measure against which other solutions can be tested, so that the economic consequences of making a non-optimal solution can be quantified. The decision maker then will know what the additional cost for meeting a political demand, or an ecological postulate will be, and it is left to his political conscience to decide if there are more important uses for the money which can be saved if such demands are not met.

The third stage is the detailed planning of the selected alternative, which is the project design. This is the solution of the problem formulated, for a single purpose and single unit reservoir in section 5 of this paper. But in a wider context, this stage must yield much more information than just the operation rules for obtaining maximum yield. It must provide information on the cost of the structures and their types, on the crops which should be grown and the irrigation methods which should be employed, and if possible it should yield a benefit-cost ratio which is high enough to make the project feasible - with all the consequences of the changes introduced by the project assessed and properly evaluated.

In stage 4 the completed project evaluation by the systems engineer is once more subjected to close scrutiny by experts and decision maker, a final design is completed on which all experts agree and of which the decision maker approves, and then in stages 5 and 6 the system is constructed and put in operation. And here the water resources expert will usually find to his disappointment that the operation rules used and the crops planted etc. do not agree well with his recommendations. It would be very useful if the reasons for this can be determined from a detailed post factum evaluation. Unfortunately, there usually is no money available for such studies, so that the experience with existing projects does not often enter into the planning of new projects. It is one of the tragedies of the well meant effort of the industrial nations of providing help to developing countries that so much of the development projects suffer from too short planning, and not enough effort in considering local factors.

And so I come to the conclusion that water resources systems planning is a process which requires all the powers of the planner: his power of grasping complicated processes and formulating them into a tractable model, his powers of persuasion and leadership in guiding a team of experts to successfully contribute to a common goal, and his humanity and compassion in understanding and considering the impacts of his projects on the people which benefit and suffer from the changes which he inflicts upon their natural environment.

7. References

Anderl, B., 1975: "Vorhersage von Hochwasserganglinien aus radargemessenem Regen". Heft 7, Mitteilungen des Instituts Wasserbau III der Universität Karlsruhe.

Binark, M., 1979: "Simultane Niederschlagsgenerierung an mehreren Stationen eines Einzugsgebietes". Heft 16, Mitteilungen des Instituts Wasserbau III der Universität Karlsruhe.

Bogardi, J., 1979: "A branch and bound algorithm to find optimal construction sequence for flood control reservoirs". XVIIth IAHR-Congress, Cagliari, Sardinia.

Cembrowicz, R.G., Hahn, H.H., Plate, E.J. and Schultz, G.A., 1978: "Aspects of present hydrological and water quality modelling". Ecological Modelling, Vol. 5, pp. 39-66, Elsevere Scientific Publishing Comp. Amsterdam.

Chow, V.T., 1964: "Handbook of applied hydrology". Mc Graw Hill Book Company.

Garcia, L.E., Dawdy, D.R. and Mejia, J.N., 1972: "Long memory stream flow simulation by broken line model". Water Resources Research, 8, pp. 1100-1105.

Garde, R.J. and Ranga Raju, K.G., 1977: "Mechanics of sediment transportation and alluvial stream problems". Wiley Eastern Limited, New Delhi.

Haimes, Y.Y., 1977: "Hierarchical analysis of water resources systems". Mc-Graw-Hill Book Company.

Hiessl, H., 1979: "Die Ermittlung der Anbaufläche einer Bewässerungsanalage in einem ariden Gebiet aufgrund genauer Wasserbedarfsfunktionen". Diplomarbeit am Institut Wasserbau III, Universität Karlsruhe (unpublished).

Hino, M., 1973: "On line prediction of hydrologic system". Proc. XVth Int. Congress IAHR, Istanbul, Vol. 4, pp. 121-129.

Hurst, H.E., 1951: "Long term storage capacity of reservoirs". Transactions ASCE, 116, p. 770.

Maass, Hufschmidt, Dorfman, Thomas, Marglin, Fair, 1962: "Design of water resources systems". Harvard University Press, Cambridge Mass.

Mandelbrot, B.B. and Wallis, J.R., 1969: "Computer experiments with fractional Gaussian noises, Part 1 - 3". Water Resources Research, AGU, Vol. 5, No. 1, pp. 228-267.

Mayer-Zurwelle, J., 1973: "Optimum release strategies for systems of flood protection". Proc. of the XVth IAHR-Congress, Istanbul, Vol. 4, pp. 205-214.

Minhas, B.S., Parikh, K.S. and Srinivasan, T.N., 1974: "Towards the structure of a production function for wheat yields with dated inputs of irrigation water". Water Resources Research, Vol. 10, No. 3, pp. 383-393.

Norero, A.L., Keller, J. and Ashcroff, G.L., 1972: "Effect of irrigation frequency on the average evaportranspiration for various crop-climate-soil systems". Trans. ASAE, Vol. 15, pp. 662-666.

Penman, M.L., 1948: "Natural evaporation from open water, bare soil, grass". Proc. Royal Soc., Vol. 193, pp. 120 - 145.

Plate, E.J. and Schultz, G.A., 1972 : "Flood control policies developed by simulation: A program". Proc. of the 2. Symposium in Hydrology, Fort Collins , Colorado.

Plate, E.J. and Treiber, B., 1979: "A simulation model for determining the optimum area to be irrigated from a reservoir in arid contries". Proc. of the III. World Congress on Water Resources, Mexico, Vol. 1, pp. 1 - 15.

Schultz, G.A., 1973: "Wasserwirtschaftliche Speicherplanung". Mitteilungen des Instituts Wasserbau III, Universität Karlsruhe.

Schultz, G.A. and Plate, E.J., 1976: "Developing optimal operating rules for flood protection reservoirs". Journ. of Hydrology, Vol. 28, pp. 245-264.

Schmidt, O. and Plate, E.J., 1980: "A forecasting model for the optimal scheduling of a reservoir supplying an irrigated area in an arid environment". Proc. of the Oxford Symp. on Hydrological forecasting. IAHS Publ. No. 129, pp. 491-500.

Schmidt, O. and Treiber, B., 1980: "Ein Simulationsmodell für Tagesabflüsse in ariden Gebieten". Wasserwirtschaft 70, Heft 1, pp. 5 - 9.

Treiber, B., 1980: "Der Einfluß des Zeitintervalls auf die Speicherbemessung durch Simulation". To be published in "Wasserwirtschaft".

Wittenberg, H., 1975: "A model to predict the effects of urbanization on wathershed response". Proc. of the Nat. Symposium on urban hydrology and sediment control.

Wood, E.F., 1978: "An application of Kalman filtering to river flow forecasting" in C.L. Chiu, editor (1978) "Application of Kalman filtering to hydrology, hydraulics and water resources". Stochastic Hydraulics Program, Dept. of Civil Engrs., University of Pittsburgh, USA.

Yevjevich, V., 1967: "Mean range of linearly dependent normal variables with application to storage problems". Water Resources Research, AGU, Vol. 3, No. 3, pp. 663 - 671.

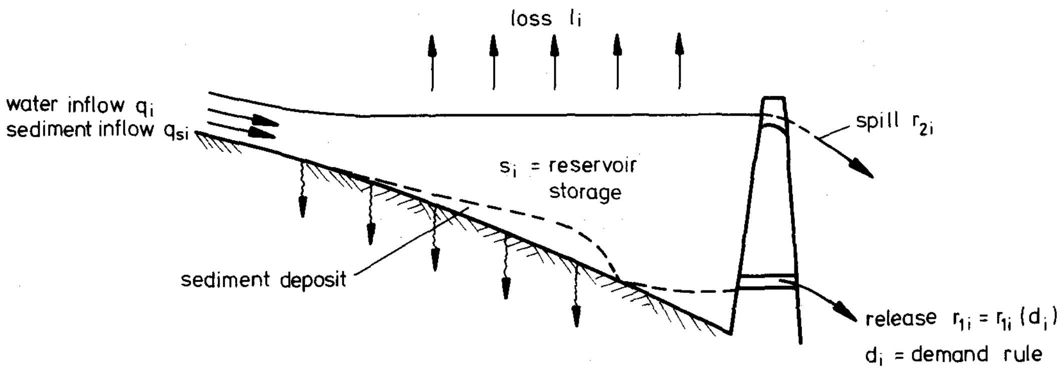

Fig. 1 : Reservoir design as a simulation problem

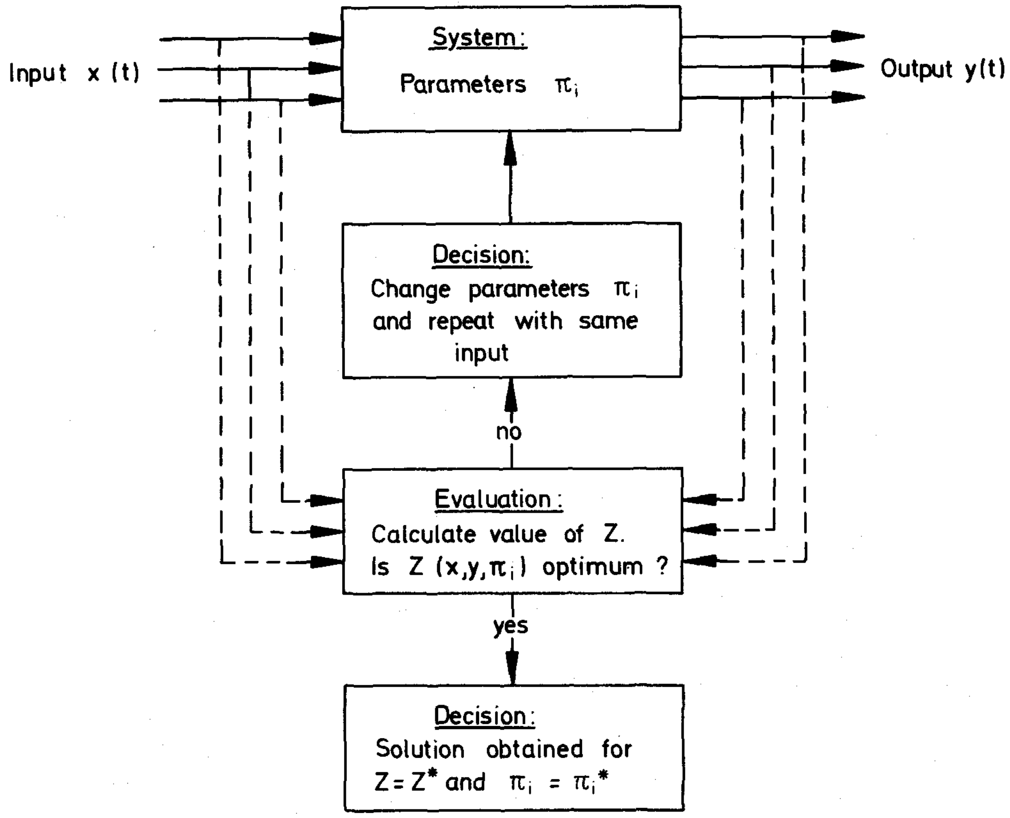

Fig. 2 : Schematics of the method of simulation with artificially generated input data x (t)

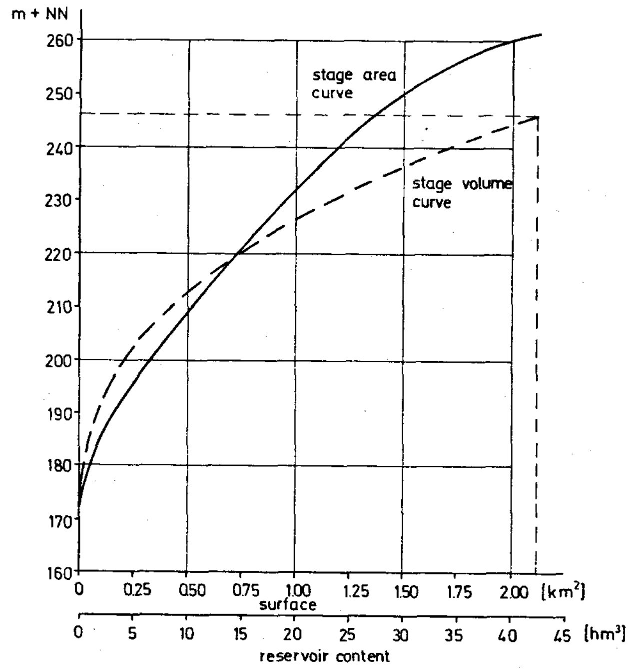

Fig. 3 : Stage-area and stage-volume curves of the reservoir

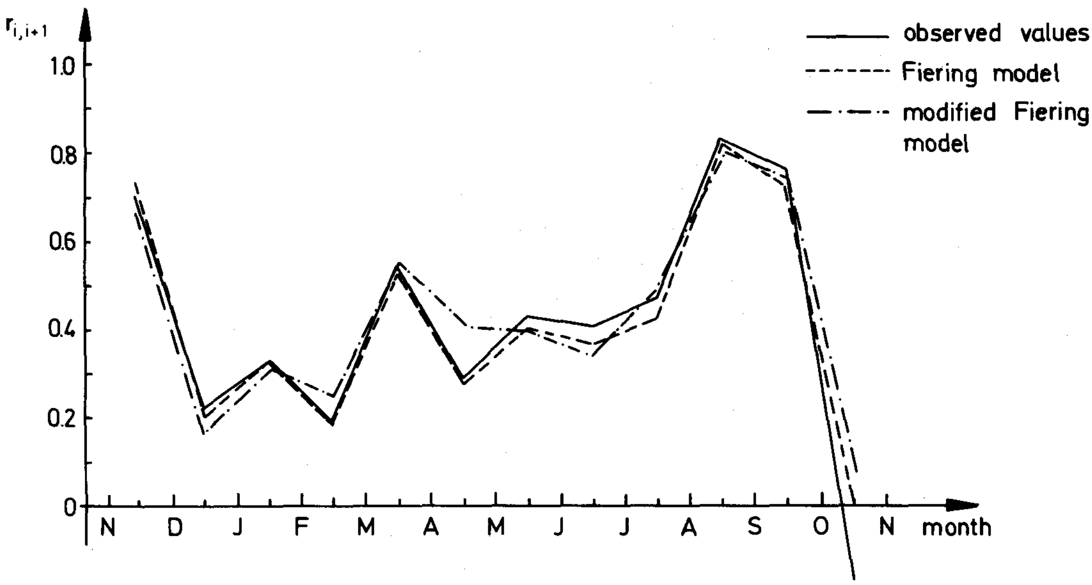

Fig. 4: Correlation coefficients for adjacent months from generated and measured data

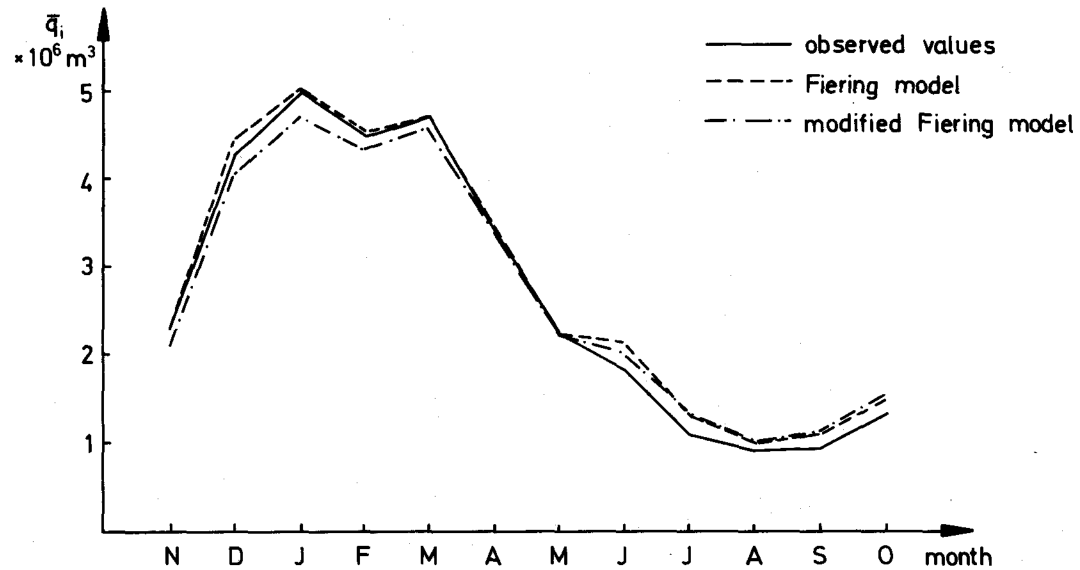

Fig. 5 : Mean monthly discharges from generated and measured data

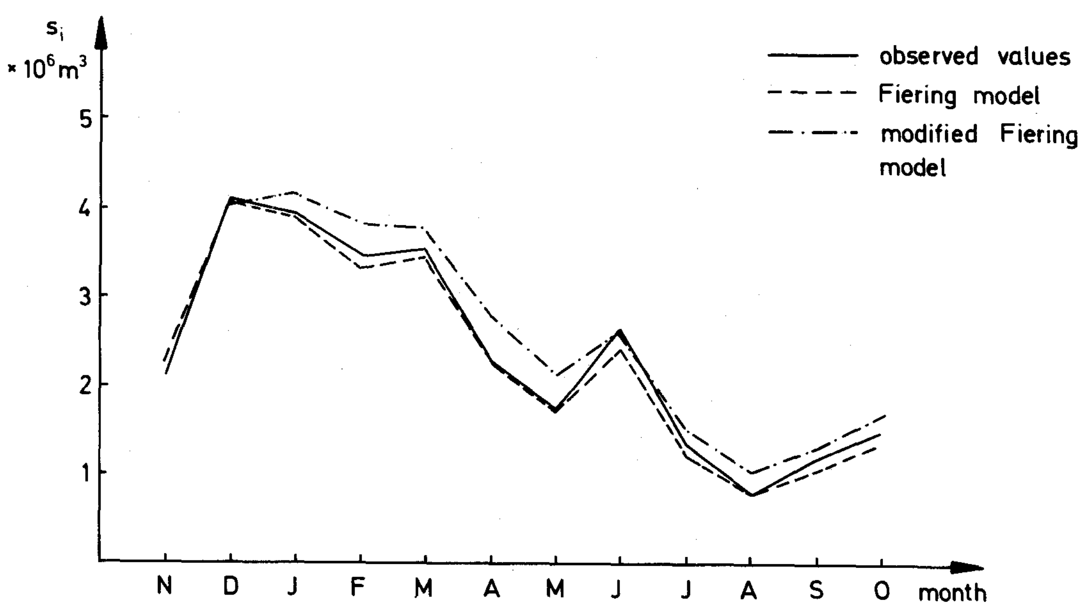

Fig. 6 : Standard deviation of monthly discharges from generated and measured data

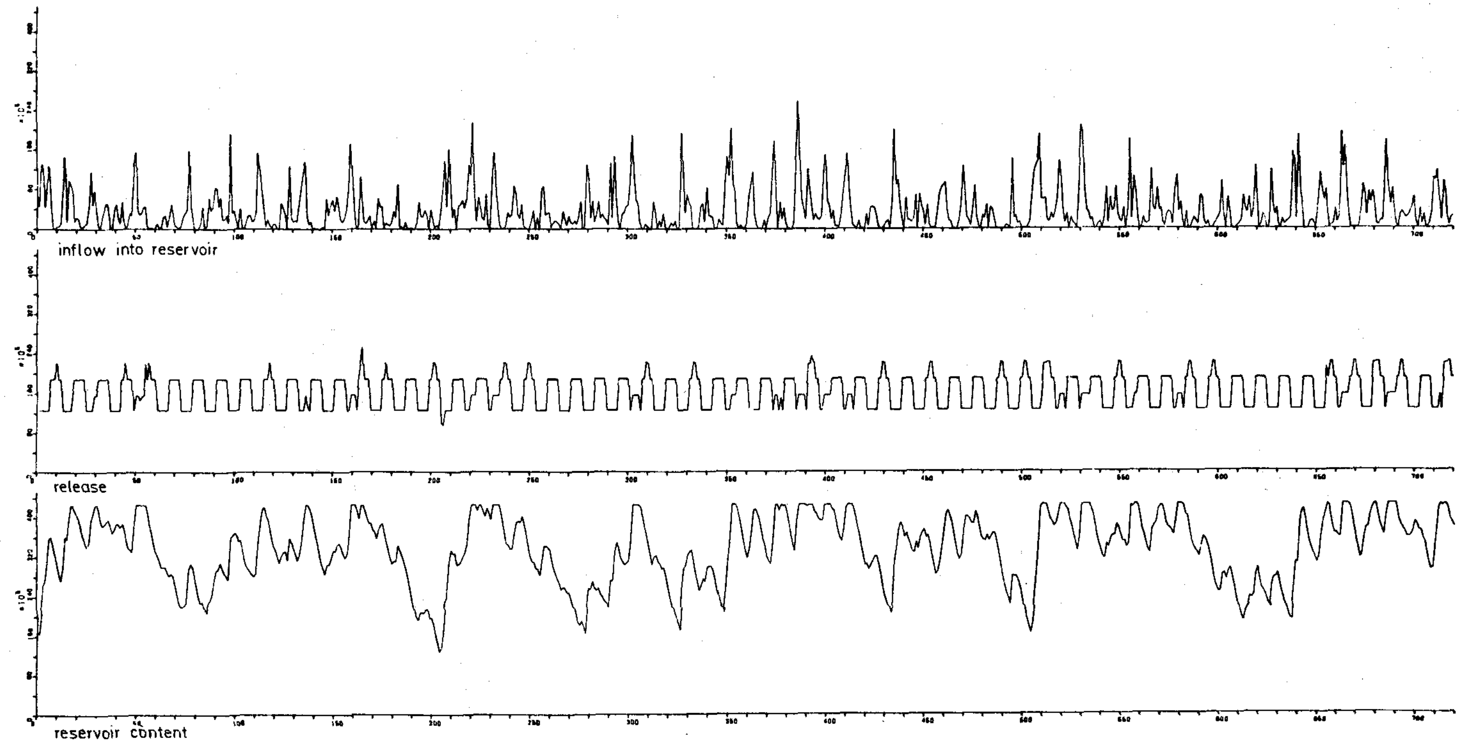

Fig. 7a : Results of simulation runs - Thomas Fiering model

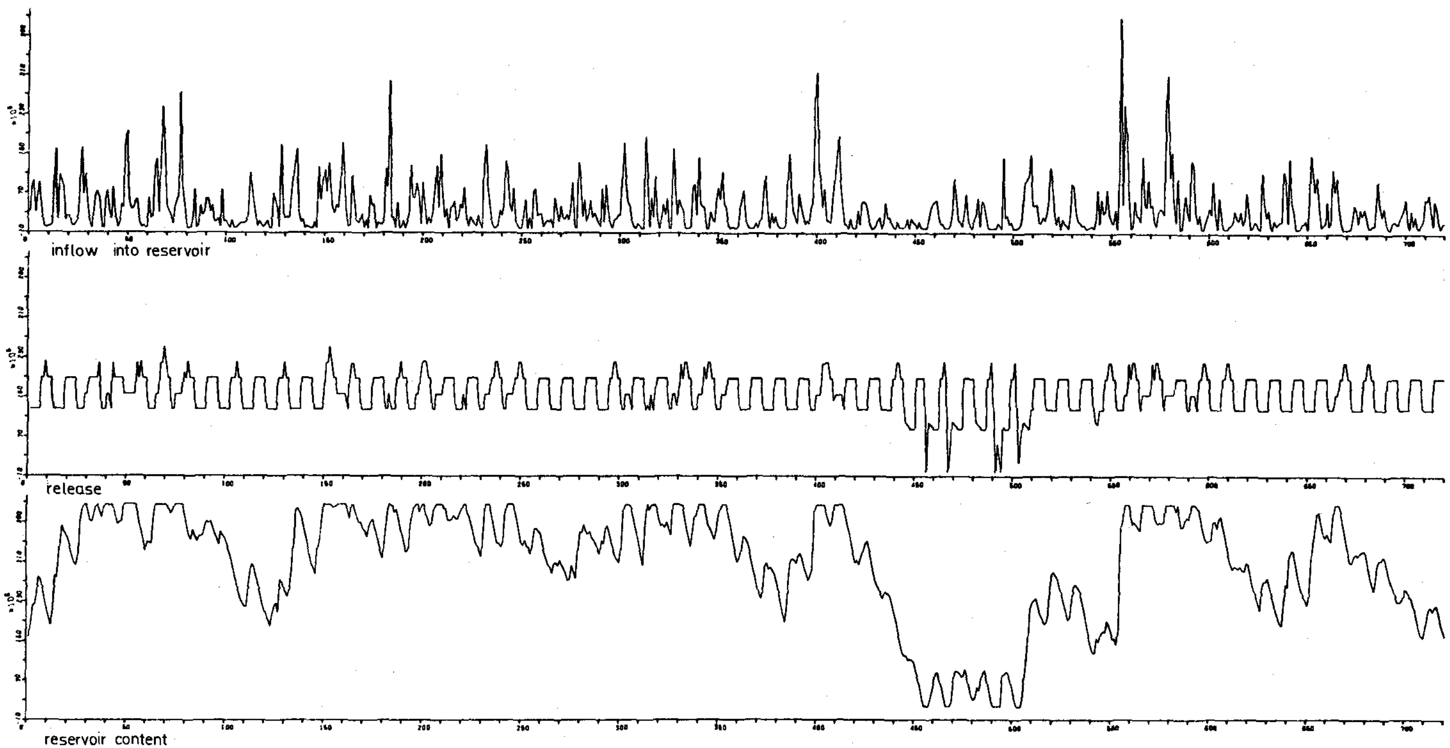

Fig. 7b: Results of simulation runs - modified Thomas-Fiering model

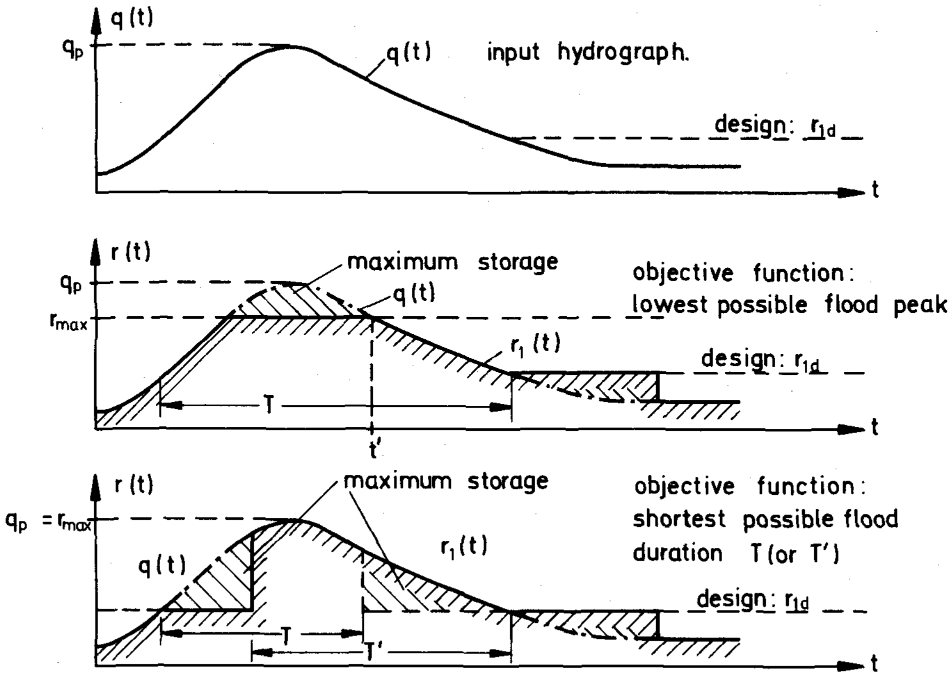

Fig. 8: Flood control policies for a single reservoir with limited storage capacity

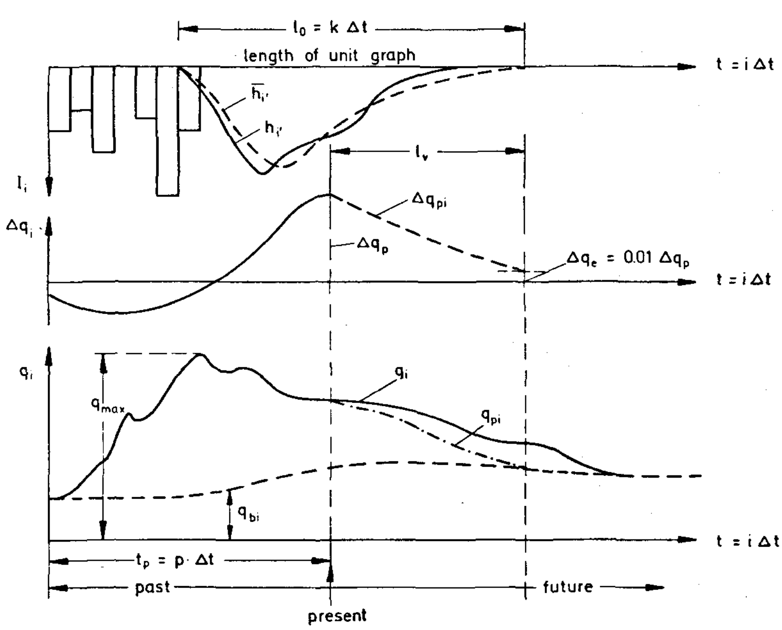

Fig. 9: Definition sketch for on-line forecasting method of Anderl (1975)

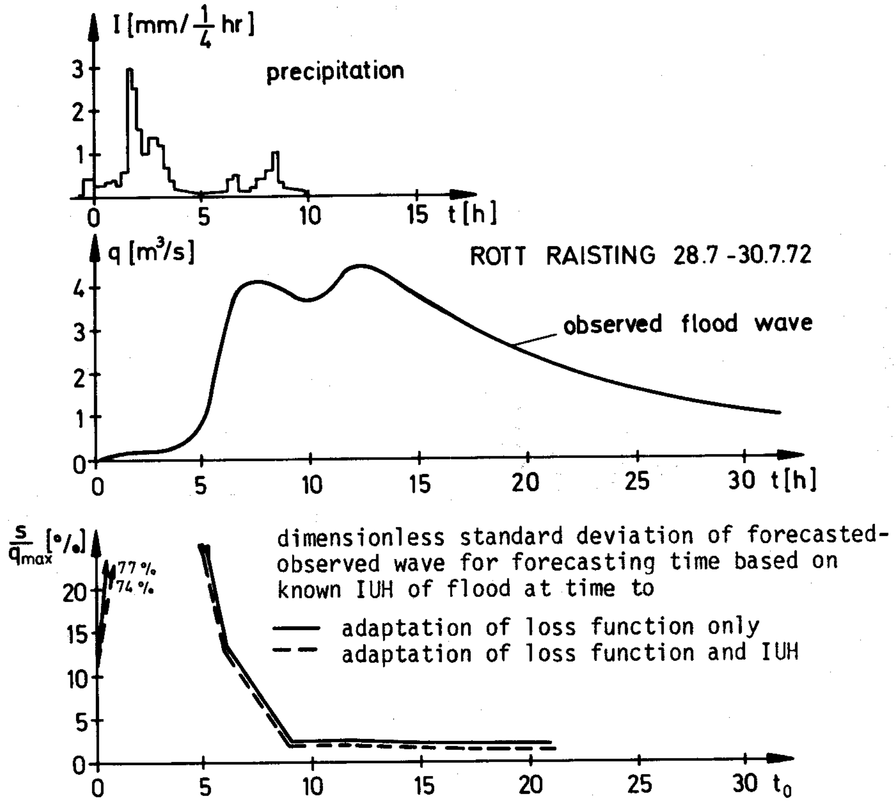

Fig. 10 : Example of forecasted wave by method of Anderl (1975): The estimated IUH is based on the same flood. Loss parameter is determined by adaptation. Forecast assumption at time t_o : for $t > t_o$: $I = 0$.

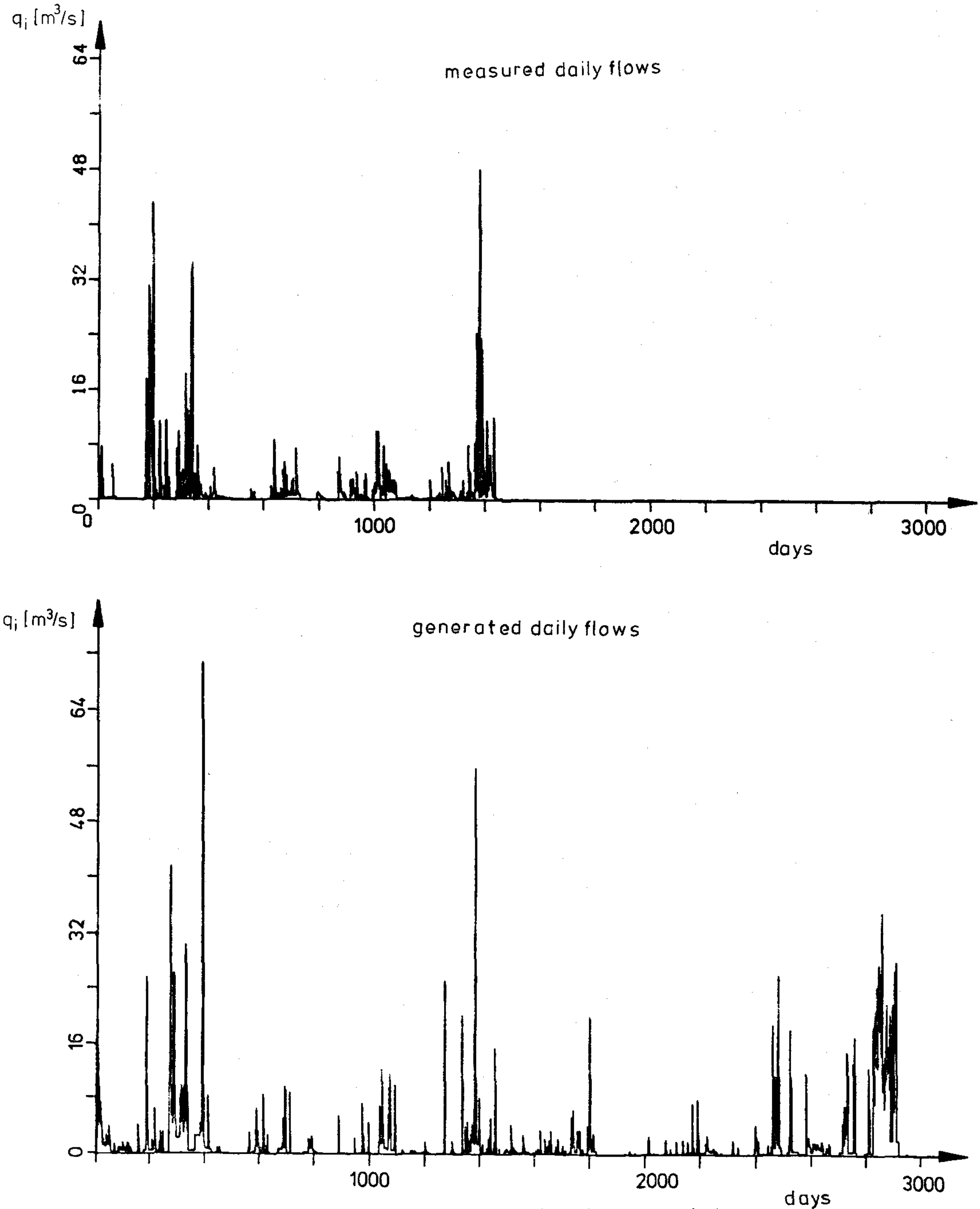

Fig. 11 : Comparison between measured and generated data

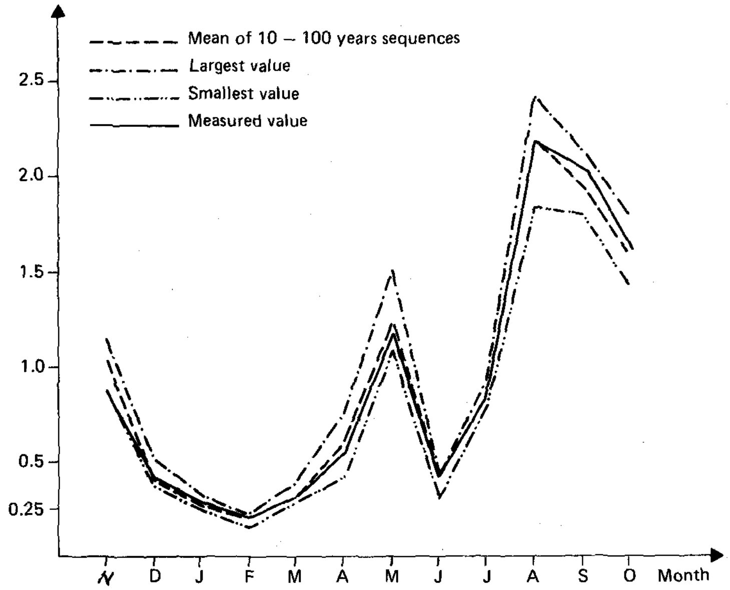

Fig. 12: Means of daily flows

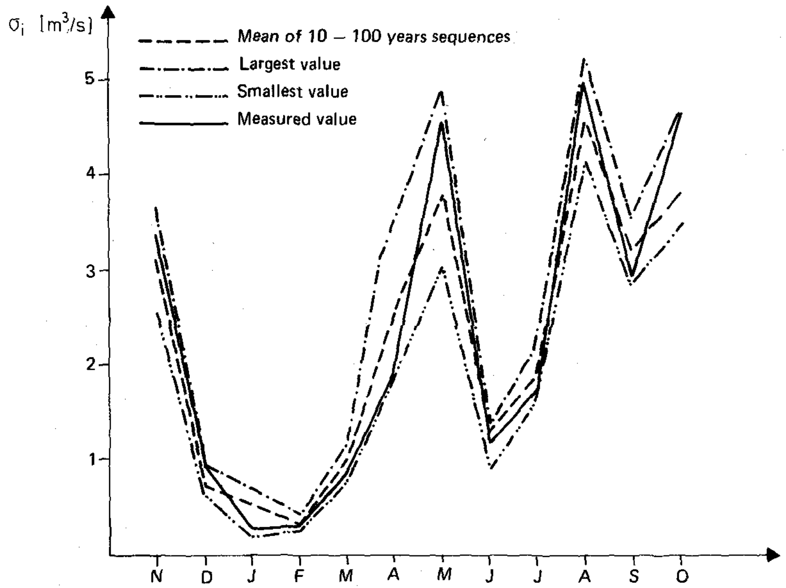

Fig. 13: Standard deviations of daily flows

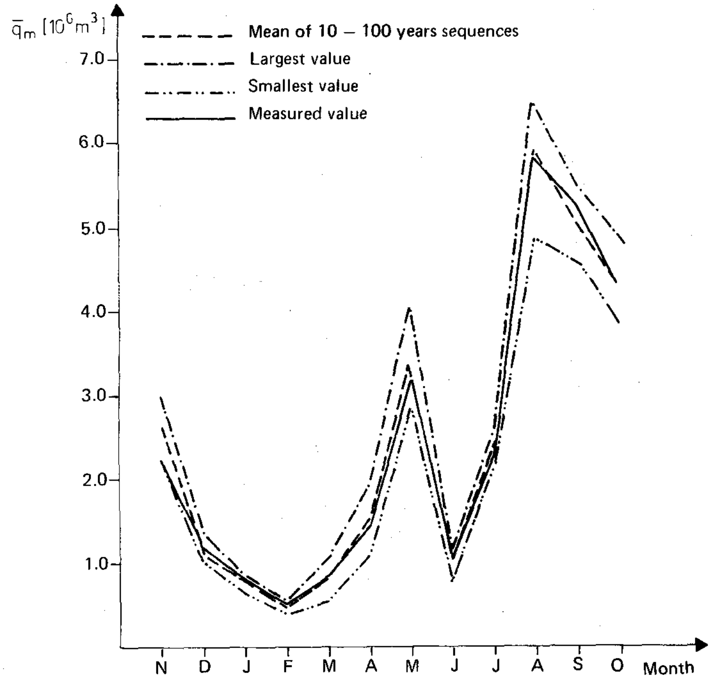

Fig. 14 : Means of monthly flows

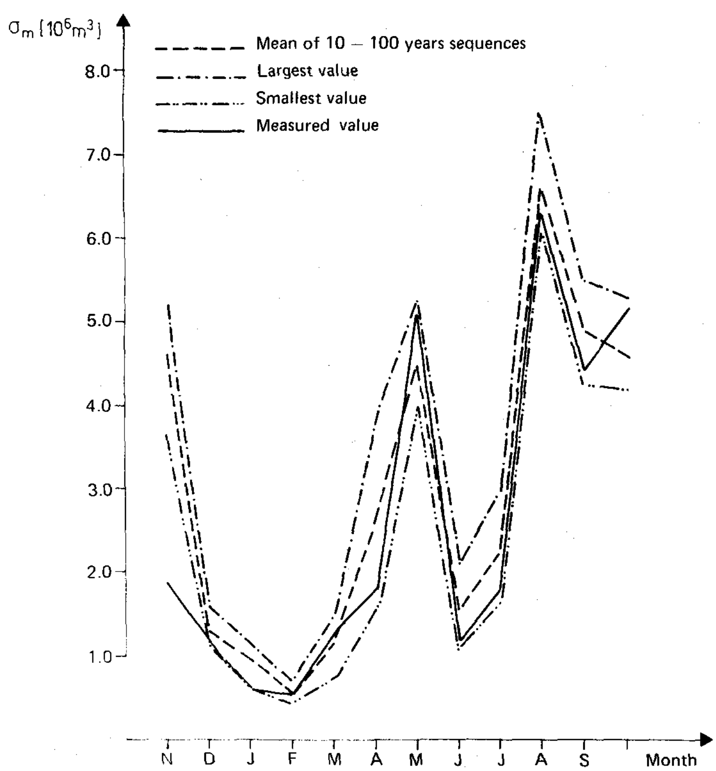

Fig. 15 : Standard deviations of monthly flows

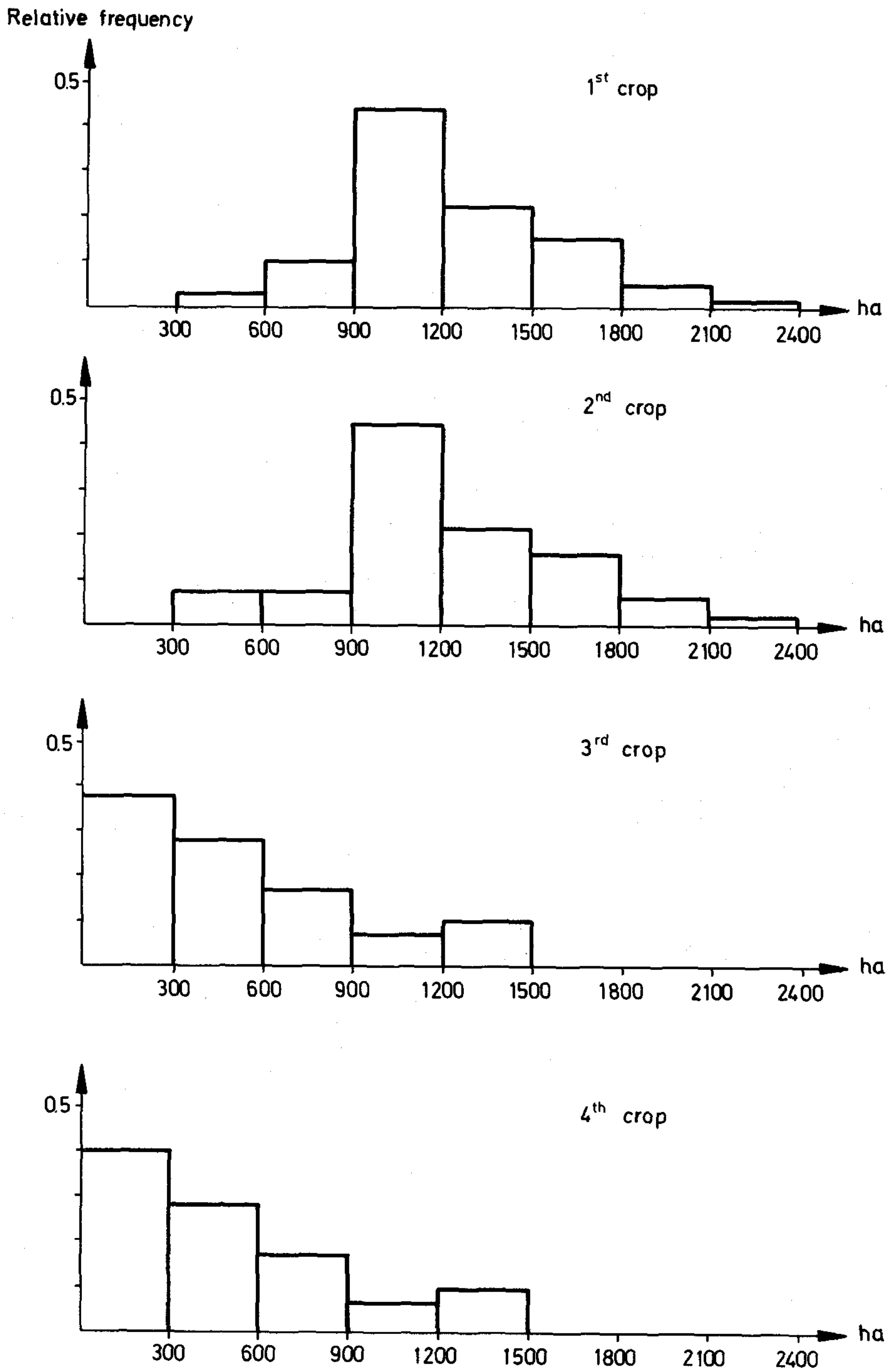

Fig. 16: Relative frequency distribution of irregeable areas for unrestricted area (divided by φ_c)

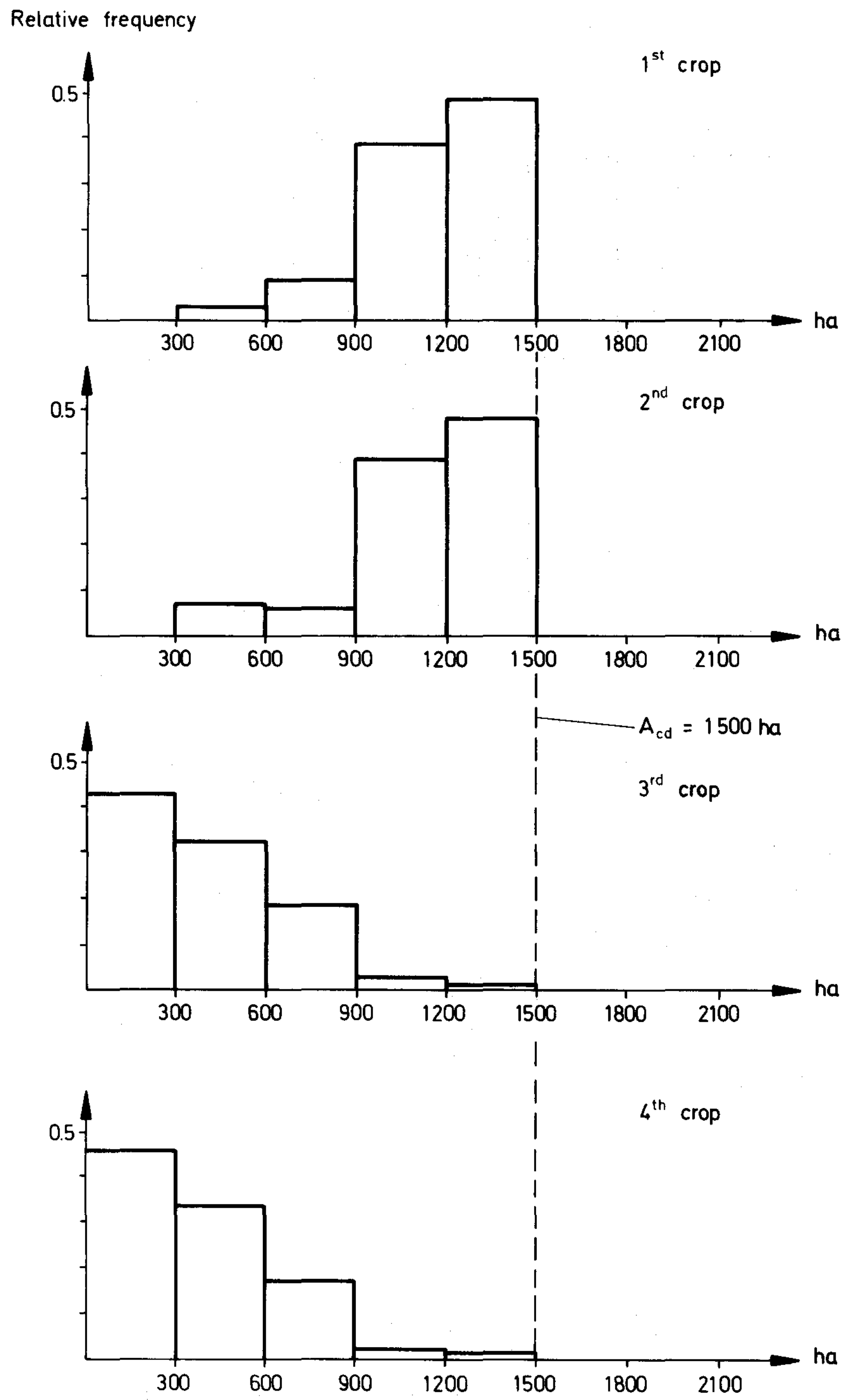

Fig. 17 : Relative frequency distribution of irregeable areas for restricted area to 1500 ha (devided by φ_c)

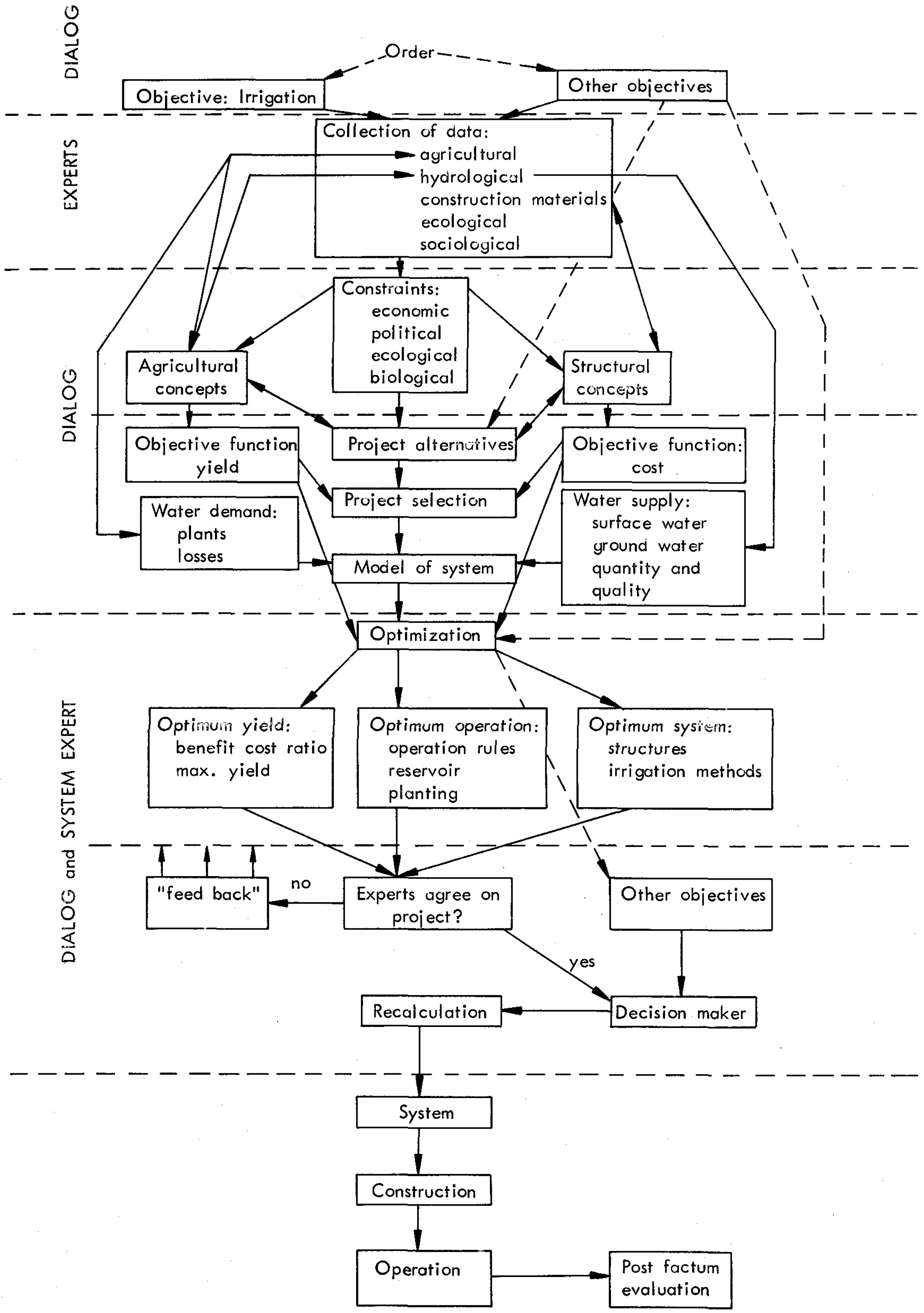

Fig. 18: General scheme of the planning process for an irrigation project