11 DATA FOR THE ANALYSIS OF FLOOD-CONTROL RESERVOIRS

The solution of the function of a reservoir that is to protect the territory along a stream downstream of it concerns both technical and economic aspects. For this reason not only technical characteristics of flood control are needed, but also data on the damage caused in the respective territories; this means that the relationship between the flood characteristics and damage must be known. Only very rarely the design reliability of flood control is used as a basis.

Flood-control release is not only reflected in the part of the stream that is to be protected, but also further downstream. Therefore, hydrological data characterizing the flood regime of the stream in the whole affected reach downstream of the reservoir must be available. If no such data are available, the calculations must be simplified or estimations must be used.

11.1 HYDROLOGICAL DATA

The time behaviour of floods in the hydrological conditions in Czechoslovakia usually has short-term characteristics—floods only last for a few days and on small streams only a few hours. To obtain a sufficiently correct pattern of a flood, the daily flows are not sufficient and a flood hydrograph must be constructed. Discharge measurements during floods are less accurate because

– the stage-discharge curve at the gauging site in the area of high water stages is less "sensitive";

– possibilities of verifying the stage-discharge curve by direct measurements during floods with a small probability of exceedance are limited;

– the relationship between the water stage and discharge during a flood due to changes in the water-level slope is not unique;

– in winter the water level may be raised due to ice-pack.

The best available hydrological data for flood regimes are chronological discharge series, e.g., of mean monthly discharges with, however, a more detailed description of the time function of the discharge $Q = f(t)$ at the time of the flood. If a flood occurs in a certain month, then the discharge should by characterized by mean daily values up to the time that the flood occurs, during the flood by continuous "point" recordings and then until the end of the month, again by mean daily discharges. If a flood occurs at the junction of two months, a detailed description of the discharge behaviour in both months must be determined. These data make it possible statistically to estimate the characteristics of the flood regime of a stream, e.g., the maximum flood discharges, flood volumes above a certain discharge value, etc.; at the same time it is possible to determine the exact state of a reservoir before a flood and therefore also the overal effect of flood-control release from it on the flood. For flood-control release the input observation period has a much greater influnce on the results of the solution (even when statistical methods are used) than for the storage function. In view of the length of Czechoslovak hydrological series, the solution of the flood-control function will probably be less accurate. The danger of underestimating maximum peak discharges in small basins was pointed out by Broža *et al.* (1978).

On the basis of the Czechoslovak Standard 73 6805, the Hydrometeorological Institute supplied the following characteristic of flood regimes:

(a) N-year maximum flood discharges for $N = 1, 2, 5, 10, 50$ and 100 years (for $N > 100$ years only if required) as one of the basic hydrological data. They are denoted $Q_1, Q_2, \ldots$, etc., generally Q_N;

(b) pattern of flood waves at the given site on the stream, expressed by the discharge curve $Q = f(t)$, i.e., by a discharge hydrograph, supplemented by digital data about the maximum peak discharge (Q_{max}) and volume (W_F); the date of occurrence is added for actually observed flood waves.

Frequently, theoretical N-year flood waves are given, the maximum peak discharge of which equals the N-year maximum discharge (Q_N) and the respective volume (denoted $W_{F,N}$);

(c) N-year flood volume above a certain selected discharge Q_x. Q_x can be the mean discharge Q_a, the mean daily discharge exceeded for 30 days in one year Q_{30d} or, e.g., Q_1, or any other suitable value. The N-year flood volume is denoted W_{N,Q_x} and it must be distinguished from the volume of the N-year flood wave $W_{F,N}$ with which it need not be in a close correlation relationship and naturally it also does not have a direct connection with the N-year maximum discharge Q_N.

11.1.1 Maximum flood discharges

Probability methods for determining the maximum flood discharges were applied in Czechoslovakia as early as 1933 (Vorel, 1933). However, only in recent years have maximum flood discharges been mapped for the whole country. The basic data used were the annual maximum discharges during the observation period. Calculations use graphical-digital methods estimating the statistical characteristics with the help of quantiles (Chap. 3).

After ascribing the probabilities of exceedance p to the respective members of a sample of observations arranged in a decreasing order of magnitude of n members, the relationship $p = m/(n + 1)$ $(m = 1, 2, \ldots, n)$, is used, which is more reliable for small probabilities of exceedance than the formula $p = (m - 0.3)/(n + 0.4)$. From among the theoretical probability distributions it is possible to choose the Pearson type III, log-normal or "exponential" (power transformation of the Pearson distribution type III, according to Kritsky and Menkel); the goodness-of-fit with the points of the empirical exceedance curve is decisive.

Figure 11.1 gives the annual maximum discharges at Bechyně on the river Lužnice (in the probability paper of normal distribution).

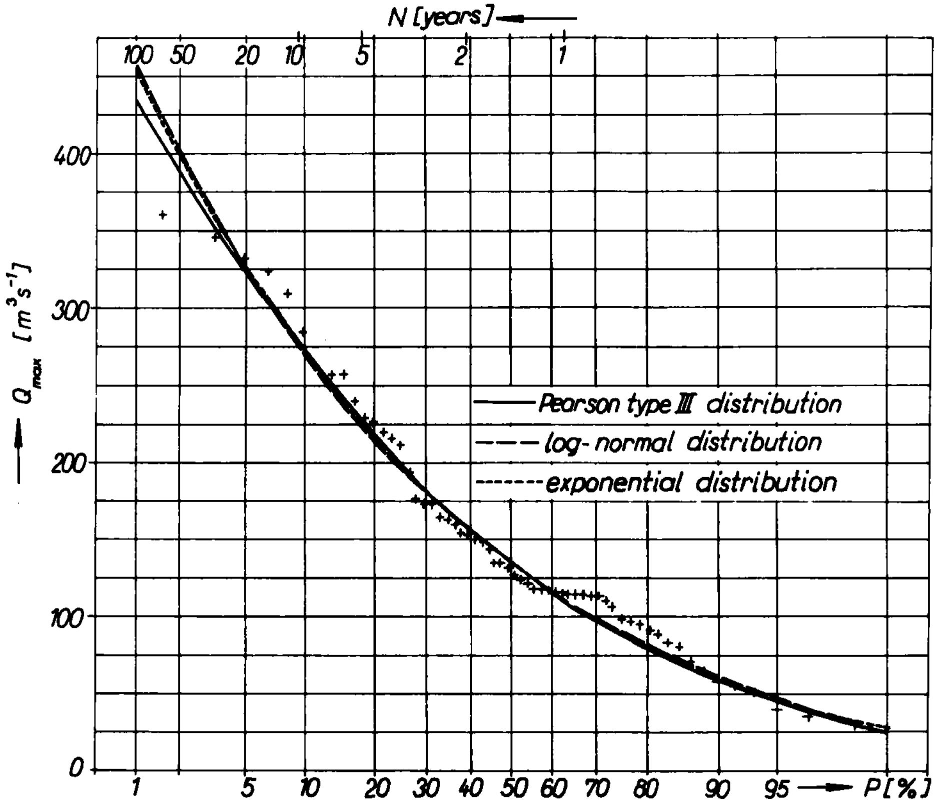

Fig. 11.1 Statistical evaluation of annual discharge maxima (Bechyně – the river Lužnice)

The probable time of exceedance N of the maximum flood discharge, currently denoted as the return period, can be calculated from the exceedance probability p, of this discharge by the relationship

$$p \doteq 1 - e^{-1/N} \tag{11.1}$$

or after rearrangement

$$N = \frac{0.434\,29}{\log(1 - p)} \tag{11.2}$$

Values of p for values $N = 1, 2, \ldots 1000$ years are given in Table 11.1

This method of determining the maximum flood discharges has practical advantages, especially for the processing of observation samples. As compared to

Table 11.1 Relationship between various forms of probabilities of exceedance (occurrence periods) of floods

N [years]	1	2	5	10	20	50	100	1000
P [%]	63.2	39.3	18.1	9.52	4.88	1.98	0.995	0.0999 5
N' [years]	1.58	2.54	5.52	10.5	20.5	50.5	100.5	1000.5

selection, which included the maximum discharges of all floods that occurred (usually a certain value was selected as the lowest limit), the sample of annual maxima is clearly defined, and no problems are caused by floods with several maximum peaks, etc.

However, it must be quite clear what exceedance probability the sample of annual maxima determines. The value p indicates the probability that in any year a certain maximum flood discharge will be exceeded, regardless of the number of times that this will occur in any one year. The time $N' = 1/p$ indicates that a certain maximum flood discharge will be exceeded on the average in the year out of N'years (or several times in that one year).

In view of the fact that the occurrence of maximum flood discharges (as well as other characteristics of floods) is usually defined as an exceedance of the maximum discharges on the average of once every N years, i.e., different from the usual probability estimation of samples of annual maximum discharges, the results must be calculated according to equations (11.1) or (11.2). That this is necessary was proved by comparing the results of the statistical estimations of the two types of observations, i.e., annual maxima and maxima of "all" floods (Dzubák, 1950 and others).

The same method can be used when estimating the maximum discharges for a certain calendar or functional period of a year, e.g., vegetation period, period of the occurrence of floods originating exclusively from rain, etc.

11.1.2 Flood volumes

The statistical evaluation of the volumes of flood waves is more complicated than the evaluation of maximum discharges. The compilation of observation samples frequently depends on the purpose the hydrological data are to serve. However, flood volumes are of foremost importance for flood-control release from reservoirs.

In the Hydrological Conditions of Czechoslovakia – part III (Hydrometeorological Institute, 1970), annual maxima of discharge volumes for various periods of time Δt (2, 5, 10 and 30 days) have been determined. The method is similar to that for maximum discharges. The best fit was attained with the log-normal probability distribution. Besides annual maximum volumes the maxima for winter and summer periods were also studied; however, these calculations of flood volumes cannot be used for the flood-control function of a reservoir.

For flood control release there is no point in working with flood volumes in the hydrological sense, but with volumes which are connected with the exceedance of non-damaging discharges. As long as these volumes exceeding the value of non-damaging discharges are held by a reservoir, the non-damaging discharge will not be exceeded regardless of the maximum flood discharge. Statistical calculations of flood volumes exceeding the given discharge value are the data most frequently used to determine the flood-control function of a reservoir.

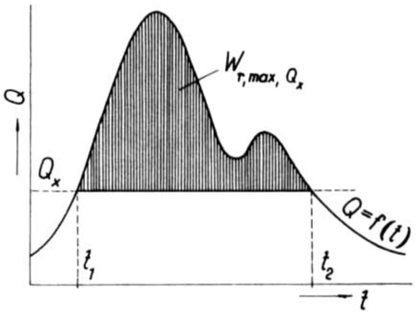

Fig. 11.2 Choice of a sample for statistical processing of annual flood maximum volumes above the discharge value Q_x

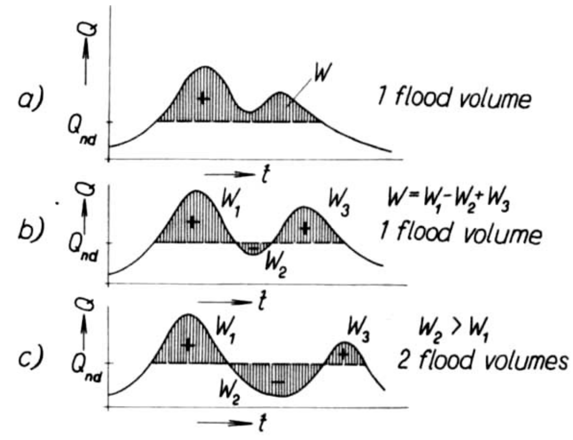

Fig. 11.3 Method for the selection of a complete range of samples of flood volumes exceeding the value of non-damaging discharge Q_{nd}

The method can be similar to that for flood discharges. In every year the largest flood volume above discharge Q_x, i.e., $W_{r\max,Q_x}$ (Fig. 11.2), is chosen and a sample of annual maximum volumes compiled. With a graphical-digital procedure, using the quantile method, a suitable theoretical distribution is fitted to determine the N-year flood volumes (bearing in mind the relationships between the probability of exceedance p and the return period—Table 11.1).

With a high non-damaging discharge $Q_x = Q_{nd}$ in some years the flood volume can equal zero. Then the sample of observations is not homogeneous and statistical estimations are difficult (e.g., the truncated probability distribution has to be considered). Another possibility is to go back to the sample of observations and include in it all floods exceeding the value of the non-damaging discharge. Uncertainties in the selection of observation samples do not concern the volumes, as floods with several maximum peaks can be estimated with good results (Fig. 11.3). In the case in Fig. 11.3a the flood volume is unambiguous. In the case in Fig. 11.3b the significant volume is given by the algebraic sum of volumes W_1, W_2 and W_3; if the absolute value of volume W_2 is bigger than W_3, the significant value of the volume is W_1. In Fig. 11.3c a case is shown in which the floods are included in the observation sample independently of one another, as the discharge depression between the waves allows for a complete emptying of volume W_1.

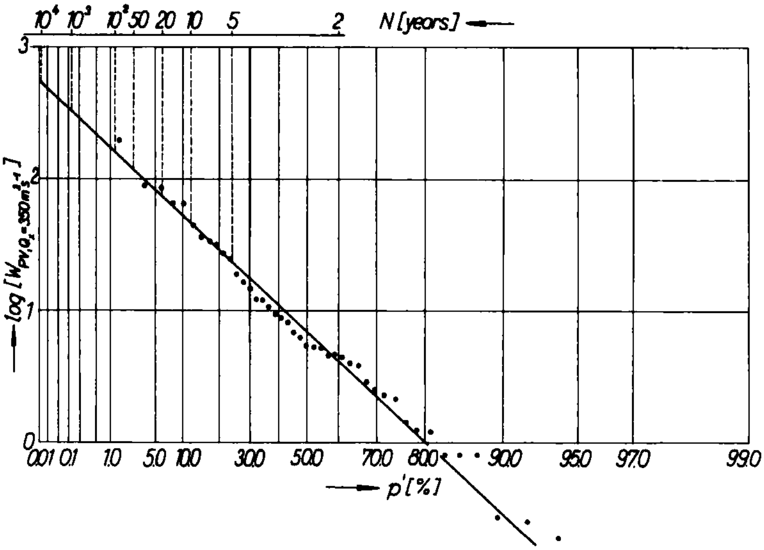

Fig. 11.4 Statistical estimation of flood volumes above the value Q_x by means of the Goodrich distribution (Berounka–Křivoklát; $Q_x = 350\ \mathrm{m^3\,s^{-1}}$)

Goodrich's exponential probability distribution (Chap. 3) is best suited for this type of sample. In Fig. 11.4 the curve of exceedance of flood volumes exceeding the discharge $Q_x = 350\ \mathrm{m^3\,s^{-1}}$ is drawn in the probability paper of Goodrich's distribution for the river Berounka at Křivoklát for the period 1887 to 1940. When stating the probable time of exceeding (return period) N flood volumes it must be considered that in this case the sample of observations reflects the real frequency of flood occurrence and a generally different number of members of the sample of observations (n') than the number of years of observations (n).

Then the probable time of exceedance N is given by the relationship

$$N = \frac{100}{p'} \nu \tag{11.3}$$

where $v = n/n'$ is the mean period of flood occurrence,
p' is the probability of exceeding the flood volume (occurrence-based).

The volume of flood waves influnced by the more complicated operation of reservoirs can be calculated similarly. However, the same operating rules have to be applied to all floods of the observation samples.

11.1.3 Model of a flood regime on a stream

At present there is no perfect model of the discharge regime which would include the related creation of floods in a large catchment. Not even a simplified model, which would include discharge series in a system of water gauging stations on the river, has so far been elaborated for flood-control purposes in many basins.

Hanzl (1971) elaborated an approximate procedure to determine the decomposition of observed floods at two neighbouring sites, which makes it possible to express the time behaviour of floods from the interbasins. This method considers the discharge-travel time between the two sites and what the effects of the inundation are between them. In compiling the theoretical flood waves from the partial interbasins, the method considers the shape and volume of the flood waves in the respective interbasins, the moment of the beginning of a flood in each partial catchment on a common time axis, the flood-travel time between the respective sites, and the changes of floods under the influnce of the inundation areas (flood routing).

Random samples are made and the composite flood is checked to see whether it is realistic. The whole samples of composite flood waves must be compiled in such a way that the maximum discharges correspond to the recurrence curve of N-year discharges.

For comprehensive solution of the control function of a reservoir (system of reservoirs), synthetic discharge series in a system of river sites should be used, reflecting the time behaviour of the discharge in such detail as to characterize the flood phenomena accurately.

Regression models encounter numerous theoretical as well as practical problems. A way of avoiding these problems is to use a combination of genetic and statistical methods. Kos (1975) considered genetic methods as more suitable for flood discharges; he derived the time pattern of the floods from precipitations based on the relation of surface runoff and precipitation. Synthetic precipitation series can then serve as basic data. Other methods can also be used where for the modelling of the discharge behaviour the transformed white Gaussian noise can be used (random process with normal distribution with a spectral density independent of the frequency); the transformation can be carried out by the application of various "filters". Another possibility is to express the detailed time behaviour of discharges during one month by a sample of special "fragments" and by joining them to the mean monthly discharges of the synthetic series. However, the problems of synthetic series with a detailed discharge time pattern have not yet been satisfactorily solved, not even for isolated sites. Generating in a system of stations is a very complicated process, as the model must also take into account the internal hydrodynamic relationships of the system, especially the discharge-travel time of the respective discharge waves.

11.1.4 Accuracy of the characteristics of flood regimes

The Czechoslovak Standard 73 6805 gives the general probable errors of maximum discharges classified according to reliability (Table 11.2). The extreme errors can be much larger. Today we are not yet able to supply general characteristics of the accuracy of flood volumes.

Table 11.2 Probable errors of N-year maximum discharges (%)

Reliability class	I	II	III	IV
$Q_1 \div Q_{10}$	± 15	± 20	± 30	± 50
$Q_{20} \div Q_{100}$	± 25	± 30	± 40	± 60

Of importance is the change of the conditions of the development of flood phenomena caused by the construction of reservoirs or other water schemes. The progress of a flood wave in a reservoir is faster than in the natural stream channel, which becomes apparent especially with a large backwater in a cascade or system of reservoirs or when several floods from larger streams flow directly into a reservoir. The original "natural" characteristics of the flood regime can then differ greatly from those that should be considered in calculating the flood-control function of a reservoir and in designing the safety devices of dams.

Full reservoirs in cascades greatly accelerate the travel time of flood waves; in the river Vltava the mean travel time in the original stream channel was, on the average, $10\,\mathrm{km\,h^{-1}}$; after the construction of reservoirs it increased to $30\,\mathrm{km\,h^{-1}}$ (Water Research Institute, Prague).

During the 20 years of operation of the Ustie reservoir on the river Orava, the discharge originally estimated as Q_{100} was exceeded several times: the probable reasons are the new conditions for the discharge-travel time and the presence of flood waves in the reservoir, especially from the rivers Bílá and Černá Orava.

Changes of the inundation areas along the streams in the reservoir catchment can also play an important role.

Flood characteristics can also be greatly influenced by the activities of man (changes in forestation, irrigation, etc.).

The new conditions must be estimated at least approximately and if quantitative estimations are not available the safety margin of the design must be raised.

11.2 RELEASE FROM A RESERVOIR

If a reservoir is designed as an independent measure to decrease flood damages, then the non-damaging discharge (safe canal capacity) in the given reach of the stream must be determined. Its size Q_{nd} is not the same for the whole reach. Non-damaging release from a reservoir is determined as the minimum of the difference of the non-damaging discharge Q_{nd} and the increment of the discharge from the interbasin ΔQ_P:

$$O_{nd} = \min (Q_{nd} - \Delta Q_P) \tag{11.4}$$

where ΔQ_P is related to the considered reliability of flood control. The size of the non-damaging discharge can change during the year, e.g., in summer it will be lower than in winter, etc.

Flood control is usually ensured by a combination of river training, channel improvement and by the flood-control function of a reservoir (or several reservoirs). We select several values of non-damaging release from a reservoir O_{nd}, from which we determine the design safe channel capacity in the respective sections of the stream. Another variable in the optimization process is the rate of the flood control reliability. The result is given by optimizing the costs (in annual values) and benefits (reduction of flood damages). If the river training must also be built for reasons other than flood control (e.g., irrigation, navigation, etc.), then only the costs for flood control are included in the technical and economic considerations.

A specific task of flood-control release from reservoirs is to hold part of the flood-wave volume at the beginning of the flood and to delay the increased discharge long enough to carry out all the necessary technical and organizational measures that help decrease flood damages. These measures can be, e.g., warning the people and even moving them temporarily from some houses, removal of ice blocks, opening of tilting gate weirs, and lessening of the effect of the encounter of flood waves from the main stream and tributaries flowing into the stream downstream of the reservoir.

Release from a reservoir during floods can depend on the position of the water level in the reservoir which determines the pressure head of the bottom outlets or the overfall depth of an ungated spillway, etc.

Then the differential equation of the reservoir flood-control function

$$P\,dt - O\,dt = dV \tag{11.5}$$

where P is the inflow
O – release
V – volume

must be integrated, taking into account the relation $O = \xi(h)$, $V = \Phi(h)$, where h is the characteristic of the position of the water level.

As equation (11.5) cannot be solved analytically, a graphical or digital integration is applied according to its differential form

$$P\,\Delta t - O\,\Delta t = \Delta V \tag{11.6}$$

which is adjusted in terms of the selected method of approximate integration; P and O are usually considered to be mean values in the interval Δt:

$$P = \frac{P_{n-1} + P_n}{2}; \qquad O = \frac{O_{n-1} + O_n}{2}.$$

From among the many solutions of this "transformation of a flood wave in a reservoir" (flood routing), we shall describe the method derived by Urban (1956) and Záruba (1961), which is very simple. Thirriot (1970) described a general method of a graphical integration of differential equations of this type.

Urban's method (1965)

As a starting point the author used equation (11.6) which he rearranged as follows

$$\frac{2V_n}{\Delta t} = \frac{2V_{n-1}}{\Delta t} + (P_{n-1} - O_{k-1}) + (P_n - O'_n) - (O_n - O'_n) \tag{11.7}$$

where O'_n is an auxiliary value which, on the right-hand side of the equation, is both added and subtracted.

Curve $P = f_1(t)$, the depth-release curve $O = \xi_2(h)$ and the reservoir depth-volume curve $V = \Phi(h)$ are given. The release time pattern $O = f_2(t)$ should be determined.

First of all, an auxiliary curve $O = \varphi_2(2V/\Delta t)$ is determined from relationships $O = \xi_2(h)$ and $V = \Phi(h)$ with a selected constant interval Δt, which is then plotted in the graph (Fig. 11.5). The scale for values $2V/\Delta t$ is the same as for P and O. Urban

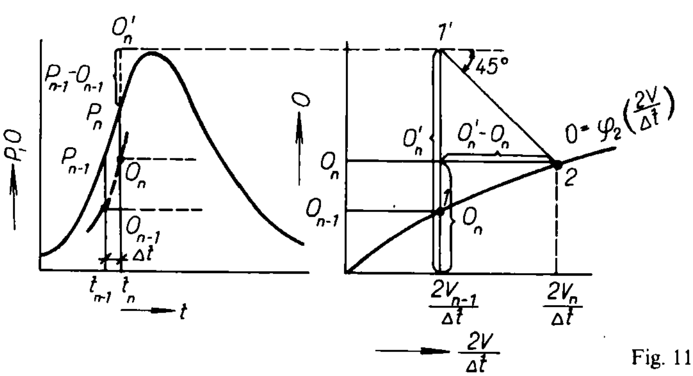

Fig. 11.5 Option 1 Urban's solution

introduced five possibilities of how to select the auxiliary value O'_n, of which we shall describe two—the first and the fourth:

1. $O'_n = P_n + (P_{n-1} - O_{n-1})$
4. $O'_n = O_{n-1}$

When using possibility 1, parts 2 and 3 of the right-hand side of equation (11.5) are omitted, as $(P_{n-1} - O_{n-1}) + (P_n - O'_n) = 0$.

The known state at the moment t_{n-1} is taken as a starting point and interval Δt is chosen. In the auxiliary graph the respective $2V_{n-1}/\Delta t$ corresponds to value O_{n-1}. At point *1* a vertical line is drawn that intersects the horizontal line with the ordinate $O'_n = P_n + (P_{n-1} - O_{n-1})$ at point *1'*. The straight line drawn at an angle of 45° intersects the auxiliary curve at point *2*; its horizontal projection to the vertical t_n gives us the sought point O_n of the release curve $O = f_2(t)$.

That the construction is correct can be seen from the graphical execution in the auxiliary graph. The straight line drawn at an angle of 45° replaces the tilting of section $O'_n - O_n$ from the vertical to the horizontal.

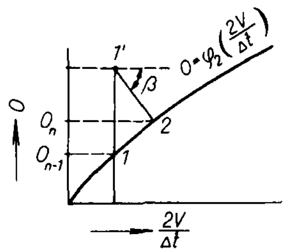

Fig. 11.6 Option 1 Urban's solution with a change of interval Δt

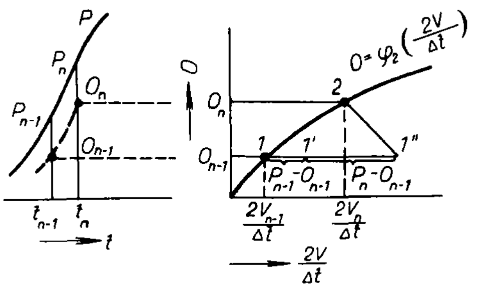

Fig. 11.7 Option 4 Urban's solution

An advantage of this alternative is that interval Δt can easily be changed during the solution. If another interval $\Delta t'$ is chosen, the straight line from point *1'* is not drawn at an angle of 45°, but at an angle β, where $\tan \beta = \Delta t/\Delta t'$ (Fig. 11.6). That this procedure is correct can be seen from the following: if curve $O = \varphi_2(2V/\Delta t)$ is drawn with the same scale for O and $2V/\Delta t$, $1\ \mathrm{m^3\,s^{-1}} = a$ [cm], then with a change of interval a change of the scale also occurs, $(2V/\Delta t') - 1\ \mathrm{m^3\,s^{-1}} = a(\Delta t'/\Delta t)$. The change of $\tan \beta$ is proportional to the change of scales.

With interval Δt, $\tan \beta = a/a = 1(\beta = 45°)$, while with interval $\Delta t'$ it is

$$\tan \beta = \frac{a}{a\dfrac{\Delta t'}{\Delta t}} = \frac{\Delta t}{\Delta t'}.$$

In possibility 4 (Fig. 11.7) a horizontal is drawn through point O_{n-1}, which intersects the curve of reduced volumes $O = \varphi_2(2V/\Delta t)$ at point *1*. Sections $P_{n-1} - O_{n-1}$ ($\overline{1\,1'}$) and $P_n - O_{n-1}$ ($\overline{1'\,1''}$) are plotted in the horizontal direction.

The straight line drawn through point *1''* at a 45° angle intersects curve $\varphi_2(2V/\Delta t)$ at point *2*, the ordinate of which gives us the sought for O_n at moment t_n.

This alternative is simple with a constant interval Δt; when using another interval a new curve $\varphi_2(2V/\Delta t)$ must be constructed.

Záruba's method (1961)

This is essentially a very simple graphical integration where the section of the depth-volume curve $V = \Phi(h)$, in which the flood wave is transformed, is replaced by a straight line. Exchanging the curve for a straight line does not significantly affect the accuracy of the graphical method. If one single straight line is not satisfactory (with a great depth of the volume in which the flood wave is held), then the curved section of curve $V = \Phi(h)$ can be replaced by broken line.

Equation (11.6) is rearranged to take the form

$$O_n = O_{n-1} + (P_{n-1} - O_{n-1}) + (P_n - O_{n-1}) - \frac{2\,\Delta V}{\Delta t} \tag{11.8}$$

The depth-release curve $O = \xi_2(h)$ is an auxiliary curve and the direction is constructed by plotting value $c(2\,\Delta V/\Delta t)$ on axis O and the corresponding $c\,\Delta h$ on axis h (Fig. 11.8). The volume increase ΔV corresponding to the height of the layer Δh, in which it is presumed that part of the flood volume will be held, can be read from the depth-volume curve $V = \Phi(h)$ (Fig. 11.8b). Constant c is a suitably chosen number, by which the calculated value $2\,\Delta V/\Delta t$ as well as Δh are multiplied to obtain the suitable size of the sections plotted on the coordinates.

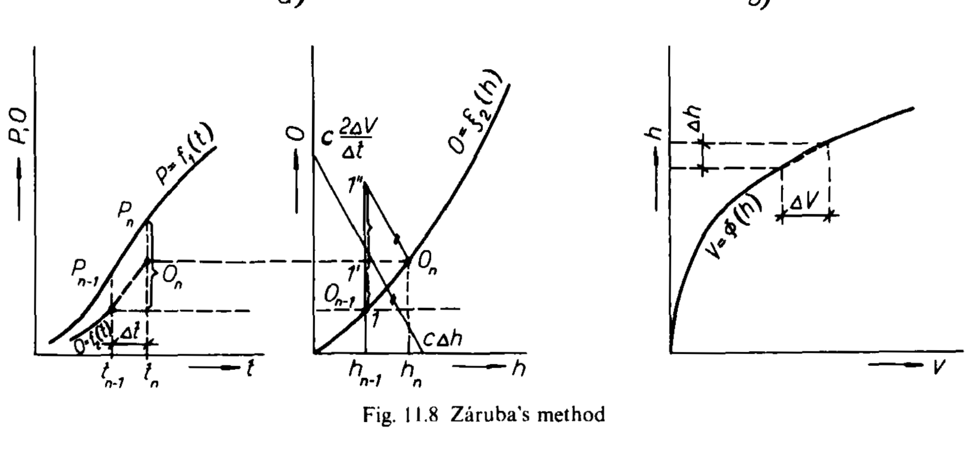

Fig. 11.8 Záruba's method

Presuming that the state at the moment t_{n-1} is known (O_{n-1} is known), another interval Δt is plotted. In point (h_{n-1}, O_{n-1}) of curve $O = \xi_2(h)$ a vertical is constructed, on which abscissae O_{n-1} are plotted. $(P_{n-1} - O_{n-1})$ and $(P_n - O_{n-1})$ are plotted one after the other. From point *1″* a straight line is drawn parallel to the auxiliary direction, which intersects curve $O = \xi_2(h)$ at point *2*, the ordinate of which gives us the sought value O_n in time t_n (Fig. 11.8a). The auxiliary direction implicitly implements the reading of values $2\,\Delta V/\Delta t$ for ΔV corresponding to the increase Δh in the time step Δt.

Another advantage of Záruba's method is that intervals Δt can easily be changed; it is sufficient to construct a new auxiliary direction by plotting section $c(2\,\Delta V/\Delta t')$ on the axis of the ordinates.

The graphical solution can be replaced by a graphical-digital solution, or only by a digital solution, when a digital computer is to be used (e.g., Water Resources Development and Construction Institute, Prague, 1970; Kazda, 1976).

The methods mentioned above are only one way of solving problems of flood control release when $O = \xi(h)$. It is important to have an accurate formulation of operational and other flow control conditions of water-management for the input data processing.

Sometimes reservoirs do not have a flood control function. In which case, the designer should not include special flood-control structures in the design; this would rise to further costs, but he should, however, for control purposes use all the possibilities given by the locality in determining the rules for reservoir operation during floods for flood alleviation.

11.3 RELIABILITY OF FLOOD CONTROL

The reliability of flood control can be expressed suitably by the probable time of exceeding (return period) non-damaging discharges and therefore of flood damage. Other quantitiative characteristics are the exceedance curve of maximum peak discharges or flood volumes transformed by the effect of a reservoir from which the exceedance curve of flood damage can be estimated.

An important part of any technical or economic analysis is the determination of the reliability of flood control; only in exceptional cases is the required return period an explicit item of input data. It is very difficult to determine the relationship between the technical characteristics of the floods and the flood damage; however, some principles have been determined for the calculation of flood damage. Another method is to base the design reliability of flood control on previous experience. According to comparable results published in the Water Management Plan of Czechoslovakia it is estimated that 60% of flood damage affects agriculture, while the remaining 40% affect other areas of the national economy. Flood-control measures on small streams are usually based on simple, mainly empirical principles.

For agricultural lands flood control is often designed for a period of 3 to 5 years (throughout the year) which ensures a 5–10 year protection in the growing period; special cultures (hops, vegetables) have a 5–10 year protection rate. A higher reliability, e.g., 20 to 50 year protection, is chosen for important communities, a hundred-year or even longer flood control is designed for large towns, important industrial plants, important roads and railways (Kabele and Plecháč, 1964).

The reservoir on the Lomnický Brook, together with the reservoir at the river Teplá, protect Karlovy Vary from floods (Fig. 11.9). In the future, the flood-control capacity of the Lomnický Brook reservoir is to be used for storage (drinking water); flood control will then be ensured by diverting flood discharges from the river Teplá to the river Ohře by a canal.

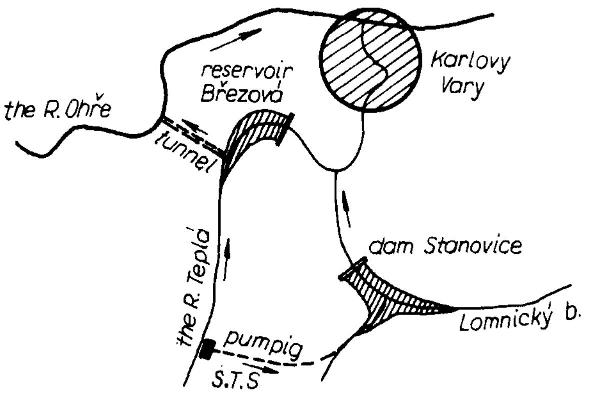

Fig. 11.9 Schematic representation of flood control at Karlovy Vary

In calculating the flood-control function of a reservoir it is necessary to have an idea of how it can influnce the stream's flood regime. An over-year release control reservoir in the upstream catchment of a river can modify floods locally relatively easily, with only a slight increase in the height of the dam; in the middle and lower parts of the stream this effect is negligible. Forecasts will also not be very useful as information, as they can be received only a short time in advance.

On the other hand, it is often very difficult to design the flood control capacity of a reservoir in the middle or lower parts of a stream which would ensure a really reliable flood control. However, as discharge forecasts are received well in advance of time, a greater protection effect can be ensured by emptying part of the storage capacity in time. Reservoir operations are practically not reflected at all in floods with a small probability of exceedance. However, within the given flood-control reliability usually the reservoirs greatly decrease the flood discharge.

11.4 PROPERTIES OF THE FLOOD-CONTROL CAPACITY

On the arrival of a flood wave the reservoir should have an empty capacity equal to (or larger than) the design dimensions of the flood-control capacity V_r. This capacity can be temporary (created by emptying part of the reservoir before the flood arives), or permanent. During the flood it must be possible to carry out all flood-control release operations; non-damaging release may never be exceeded without intention. The spillway and outlet devices of a reservoir must be placed so as to meet the requirements of the flood-control capacity.

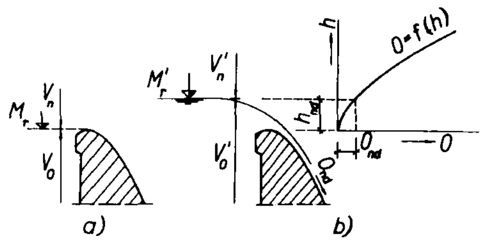

Fig. 11.10 Difference between the controlled and uncontrolled capacity from the point of the terminological standard and the function

Flood-control capacity can be defined as that part of the total capacity which serves to catch water from floods and to transform flood waves. The flood-control capacity can be controlled or uncontrolled, or partly controlled and partly uncontrolled. The boundary between a controlled and uncontrolled reservoir capacity is the lowest level of the spillway crest of an ungated spillway or the level of the upper crest of a controlled spillway gate. However, even an uncontrolled capacity to a certain level M'_r will ensure that non-damaging release is not exceeded (Fig. 11.10).

Various capacities of a reservoir can play their part in flood-control release. However, for flow-regulation analysis it is best to divide the control effect of the storage capacity from the controlled and uncontrolled flood control capacity.

11.5 FLOOD-CONTROL RELEASE METHODS

The decrease of the flood discharges to the value of a firmly determined non-damaging release O_{nd} (with a given reliability) with the help of a flood-control capacity used only for this purpose is a reliable method, one which does not require any detailed hydrological data; however, it is the most expensive method. If we are unable to estimate the control effect of the storage capacity (using the rule curve and flood forecasts), a "safe design" of flood-release control is justified.

If it is at all possible to improve the flood control effect of a reservoir by expediently combining the storage and control function, we always try to do so.

In the conditions pertaining e.g. in Czechoslovakia the management of flood-control release has a short-term character. From the point of view of the working cycle it is therefore difficult to adjust it to the storage function, although the two basic functions have many common traits.

If the part of the stream that is to be protected against floods starts further downstream of a dam, the flood-control release can be analogous to the river flow regulation. Release from a reservoir is adjusted to the pattern of the uncontrolled flood discharge from the interbasin between the reservoir and the land to be protected so as not to exceed a non-damaging discharge. Indispensable for this type of flood control is the forecast of discharges, bearing in mind the discharge-travel time downstream of the reservoir.

Flood-control release in a cascade of reservoirs can be solved in the same way as the storage function. However, it is very important to optimize the size of the storage and flood-control capacity in the respective reservoirs of a cascade and to select the most effective operation rules.

The development of flow regulation systems is reflected in the further cooperation of reservoirs of larger catchments. An example of this is the system of flood control reservoirs on the river Stropnice (Randák, 1972).

Just as for water supply, diversion can also be applied to flood control (Fig. 11.11). Figure 11.11a shows two flood-control reservoirs. As the volume of reservoir *1* is not large enough, part of the flood wave has to be diverted to reservoir *2* which

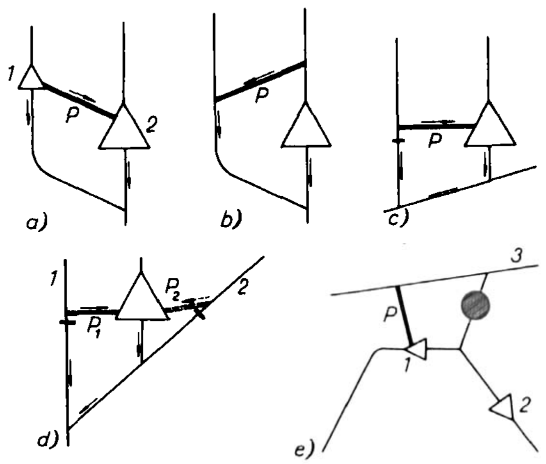

Fig. 11.11 Flood-control systems using water diversion

is in the adjacent catchment. The capacity of the diverting canal does not have to be very large, due to the effect of reservoir *1*. The man-made canal between two neighbouring streams and its effect on the flood control within a simple system with a reservoir is shown in Fig. 11.11b.

Fig. 11.11c and d show a schematic representation of two systems, with the flood-control capacity in lateral reservoirs. In Fig. 11.11d flood discharges are also diverted from stream *2*.

The schematic representation in Fig. 11.11e shows how a town with important buildings can be protected against floods. Reservoir *1* was not able to ensure the required high reliability and therefore, when another reservoir *2* was built to supply drinking water to the town, further flood control capacity was included in the design. It is presumed that with increasing demands on drinking water, the reservoir capacities will have to be redistributed. For flood-control purposes, water will be diverted from reservoir *1* to stream *3*.

The examples mentioned above prove that many different measures can be used for flood-control release to ensure the required reliability, depending on local conditions, just as for water storage.