A RESERVOIR FUNCTION AND BASIC TOOLS FOR ITS ANALYSIS

1 BASIC FUNCTION OF WATER RESERVOIRS

A water reservoir is an enclosed area for the storage of water to be used at a later date; it can also serve to catch floods to protect valleys downstream of it; to establish an aquatic environment; or to change the properties of the water. A reservoir can be created by building a dam across a valley, or by using natural or man-made depressions. The main parameters of the reservoir are the volume, the area inundated and the range that the water level can fluctuate.

1.1 FUNCTION AND SIGNIFICANCE OF RESERVOIRS IN WATER MANAGEMENT

The basic function of an artificial reservoir is to change the rate of flow in the stream, or to store water for more expedient use. Reservoirs are among the more useful means of controlling the natural character of water flows, instead of depending on nature.

Active storage (accumulation, conservation) reservoirs help to overcome dependence on the natural hydrological regime of a territory during droughts. If the water consumption in the given area exceeds the water yield, and if the building of reservoirs does not suffice to meet the demand for water, then water has to be diverted to the area of shortage from another catchment, or better use must be made of the existing water. These measures can be economically effective even before the demand for water reaches the point of safe yield.

The territory can be protected against floods by increasing the bankfull discharge, or by increasing the reservoir volume to hold the water surplus. These reservoirs are known as *flood control (retention) reservoirs*. The water can also be diverted to another catchment area.

Impounding reservoirs on streams usually serve a dual purpose: water supply and flood protection. A particular part of each reservoir serves to carry out one or other of these tasks which can be partly complementary.

The water accumulated in *conservation reservoirs* can be withdrawn for various purposes: irrigation, industrial, agricultural and public water supply, power generation, maintenance of navigation depth, dilution for sanitary and other purposes.

A proper aquatic environment, once established in the reservoir, can serve for recreation purposes and water sports; fish and duck farming; cultivation of aquatic plants; treatment of the water; improvement of the natural environment.

If a reservoir serves several purposes, it is called a *multi-purpose reservoir*. Large impounding reservoirs are usually of this type. Even if the reservoir is a single-purpose one, it usually also serves for flood control.

The integral task of a reservoir is to change the flow regime of the stream. Therefore a water volume created by a weir, the only purpose of which is to raise the water level, is not considered to be a reservoir. Such a volume is called a *backwater reach* (i.e. a volume created by weirs to increase the navigational depth or to facilitate the direct withdrawal of water from the river). However, backwater reaches created by weirs, which change the flow regime, are also considered to be reservoirs.

There is significant interaction between reservoirs and the environment: the reservoirs affect the environment and vice-versa. This is true for both the quantitative and the qualitative aspects of the water. The effect of a reservoir on the quality of the water downstream of the dam (pollution, temperature, etc.) and the impact of human activities on the quality of the inflowing water and on the mechanical, chemical and biological processes in the reservoir make the qualitative parameters of the water just as important as the quantitative parameters.

A reservoir can accomplish its function as an isolated unit, if it is the only reservoir in the system "water source – user", or it can be one of the elements in a system of reservoirs, which cooperate mutually in the water management of a given region. These days, with increasing frequency reservoirs form systems, including older ones which were originally built as single reservoirs.

1.2 THE DEVELOPMENT OF WATER RESERVOIRS

The construction of reservoirs goes back to the most ancient cultures. In the days of slavery when, compared with other crafts, construction was of a very high standard, noteworthy structures were erected, including dams and water mains.

Remnants of the first rock-fill dam, built in 3000 B.C., still stand, 30 km south of Cairo. It can be presumed that earth dams were built even earlier. Another reservoir in Egypt, Moeris, was built in about 2900 B.C. as a lateral reservoir of the Nile with a volume of $12 \cdot 10^9$ m^3 (Tölke, 1938).

Reservoirs can also be traced a long way back in India; on the outskirts of Madras alone, there were about 50 000 reservoirs. King Solomon built a system of reservoirs near Jerusalem as far back as the 10th century B.C.

Ancient Persia (Iran) could not have reached its high cultural standard without irrigation. In the 6th century B.C., the most important reservoir was Bend-e-Ramdjerd, near Persepolis, on the river Kor. It was reconstructed many times and to a limited degree is still in operation. About 500 B.C., Dareios I built "bridge weirs" on this river with many outlets and probably with wooden stoplogs, to supply water to the irrigation system (Kuros, 1943). The reservoirs Saveh (about 40 m deep) and Bend-e-Emir with gravity dams and the reservoir Kabar with an arch dam are about 1000 years old; the last two are still in operation. Over the years, their role changed, due to silting, elevation, damage or catastrophe. This also applies to the reservoirs of the turn of the 15th and 16th centuries.

At that time, reservoirs for irrigation were also built in Spain (Almanza, Alicante and Tibi) with masonry gravity dams which were sometimes up to 40 m high. The Proserpina reservoir in Spain goes back to the time of the Romans. Lake Fucino was used as the source of the great Roman water main built in the first half of the 1st century A.D. by the Emperor Claudius.

During feudalism, Europe was divided into numerous small states. These rather poor countries did not have the means to build any large constructions, and any reservoirs used were only of the pond type.

1.2.1 Construction of reservoirs on the territory of present-day Czechoslovakia

a) The ponds of Bohemia (up to the end of the 16th century)

On the territory of present-day Czechoslovakia, the first constructions built to hold water were ponds for fish farming.

The building of ponds was greatly developed in the 13th–16th centuries, thus establishing the tradition of water structures and water management.

The largest system of ponds was located at Třeboň, on the estate of the Rožmberks, founded by Josef Netolický Štěpánek (who died in 1538); the builder of the largest ponds was Jakub Krčín of Jelčany and Sedlčany (1535 – 1604). However, the pond-building tradition in that area was already 200 years old by then. Štěpánek built the 45 km long Golden Channel (Zlatá stoka) and Krčín included a new element in the system – he re-routed the flood discharges of the Lužnice through the New River (Nová řeka) to the Nežárka to protect the Rožmberk pond on the Lužnice (surface area 8.6 km^2, depth 11.5 m and volume $6 \cdot 10^6\ m^3$; however, during the flood of 1890 it held a volume of $50 \cdot 10^6\ m^3$) from the flood waters.

Pond building and fish farming at that time were described by Vilém of Pernštejn; Jan Skála of Doubrava (1480 – 1553) wrote "Five Books about Ponds" ... in Latin which were translated into English, Polish and German and published by the Czechoslovak Academy of Sciences (Dubravius, 1953) in 1953.

Table 1.1 Small reservoirs and ponds in Czechoslovakia in 1970

Index	State in 1970	Projects
number (thousand)	23 720	936
volume (mil m^3)	516.7	119.3
area (km^2)	548.9	50.6

Besides their use for fish farming, ponds help to balance the flow, but they only play a significant role if part of a larger system. In recent years, some of these ponds have been renewed and new ones built to cover the increasing needs of agriculture and to protect the environment.

(b) Reservoirs for mining purposes (up to the 19th century)

The need for water and hydro-power for the mining and processing industries gave rise to the second stage in reservoir construction in Czechoslovakia.

Here, too, ingenious systems were constructed, some of which still exist, for example, in the Krušné hory and the Slavkov forest (which date back to the 16th century) in the neighbourhood of Banská Štiavnica and Příbram (dating back to the 18th century), as well as others elsewhere. Mine timber was floated on some of the canals connecting the reservoirs, for example on the Slavkov canal (built in the second half of the 15th century) which was later extended to a length of 24 km (Majer, 1975).

The reservoirs in the neighbourhood of Banská Štiavnica have a total volume of $7 \cdot 10^6$ m^3 and are notable for the height of their earth dams, the Rozgrund reservoir (1744) being the highest, at 30 m.

(c) Reservoirs for floating timber (up to the end of 19th century)

A flood wave, caused by discharging the water stored in the reservoir, was used for floating timber.

These small reservoirs with earth dams were mainly constructed in the Šumava region (on the rivers Vydra and Křemelná). The reservoir in the Korytnická valley, a unique structure built in 1882, demonstrates the first use of rubble masonry (16 m high).

(d) Flood control reservoirs (end of 19th and early 20th century)

After the great floods of 1890 and 1897, special flood control reservoirs started to be built. Their sole task was to contain the water during floods.

The first reservoir to be built was on the Jevišovice rivulet, near Jevišovice, Moravia. Completed in 1897; it has a curved gravity dam, 25 m high, of masonry and cement mortar. In the Nisa catchment in north Bohemia there are five flood-control reservoirs built between 1902 and 1909, they are all rubble masonry gravity dams. Their total volume amounts to $6 \cdot 10^6$ m^3; the dams are 15.8–23.5 m high.

By 1918, another 11 reservoirs had been built, mainly to control floods; of the 17 reservoirs built between 1919 and 1945, over half are flood-control reservoirs; but none was built after 1945.

Flood-control reservoirs have been in operation for 50 to 90 years; however, many were later turned into conservation reservoirs. The new demands on water, better hydrological data and the advances in science make us regard the older reservoirs from a new angle, such as the safe yield, etc. Thus a better economic effect can be reached without further capital investments.

(e) Hydro-power reservoirs

The first hydro-power reservoir in Czechoslovakia was built on the Želivka near Sedlice, with a volume of $2 \cdot 10^6\ m^3$ and a power plant of 2.36 MW.

Of the reservoirs built in the period up to 1945, only the Vranov reservoir has a volume ($122.5 \cdot 10^6\ m^3$) which can effectively influence the runoff from a larger catchment area.

Large reservoirs capable of affecting runoff were only built after 1948, mainly for hydro-power. These reservoirs include the ones at Ustie on the river Orava and at Lipno, Slapy and Orlík on the river Vltava with storage capacities of between 270 and $704 \cdot 10^6\ m^3$.

These capacities not only ensure a large output from the power plants, but for Lipno and Ustie, an over year discharge control the favourable effect of which is also reflected in all the power plants down river.

Although these reservoirs were built mainly to produce electricity, they are in fact multi-purpose reservoirs which have a beneficial effect on the flow regime of the respective catchment areas.

(f) Conservation reservoirs for public water supply and industry

After the three reservoirs built at the beginning of the century: Jezeří, Kamenička and Janov, only one more conservation reservoir was added in the period between 1918 and 1945, i.e., on the Fryšták rivulet near Gottwaldov. After 1945, reservoirs were mainly built for waterworks.

Of 35 reservoirs built between 1945 and 1960, 20 were for public water supply, two for agriculture and 13 for power generation. Of the 57 reservoirs built between 1961 and 1967, 42 serve for public water supply. Their total storage capacity comes to $1317.2 \cdot 10^6\ m^3$.

According to the Czechoslovak Water Management Plan, the number of reservoirs used for waterworks is due to increase by a factor of three by the year 2000 as compared to 1970.

(g) Agricultural reservoirs

One of the primary aims of the oldest conservation reservoirs was to accumulate water for irrigation. In Czechoslovakia, irrigation water was at first withdrawn from ponds. The first irrigation reservoir was built in 1939 near Husinec on Blanice.

Table 1.2 Water demand for agriculture in Czechoslovakia

Index	Specific Unit	Year 1970	1985	2000
annual withdrawal for irrigation in a design dry year	$10^6\ m^3\ yr^{-1}$	100.1	1 133	2 257
annual water demand for livestock breeding	$10^6\ m^3\ yr^{-1}$	116	145	161

Table 1.3 Water resources and reservoirs in Czechoslovak catchments

Territory Catchment	Long-term average annual precipitations [mm]	 [mil. m^3]	Annual long-term average outflow [mil. m^3]	Reservoirs in 1970 number	 capacity [mil. m^3]	 by the year 2000 number	 storage capacity [mil. m^3]	 accumulation rate in catchment [%]
Czechoslovakia total	690	89 000	27 740	185	3569	310	6127.7	22.1
I. Upper and middle Labe	706	10 145	3 340			37	678.8	20.3
II. Vltava	657	12 015	3 353			57	1223.0	36.5
III. Berounka	594	5 509	1 257			22	339.4	27.0
IV. Lower Labe	657	6 275	2 105			33	473.2	22.5
V. Odra	825	5 158	1 953			19	310.3	15.9
VI. Morava	641	13 528	3 140			67	1016.0	32.4
Bohemia and Moravia total	668	52 630	15 148	159	2504	235	4040.7	26.7
VII. Danube	582	3 138	355			6	23.0[1])	6.5
VIII. Váh	812	13 627	5 481			30	1041.0	19.0
IX. Hron	747	9 199	2 918			16	280.0	9.6
X. Bodrog	718	10 406	3 838			23	743.0	19.4
Slovakia total	743	36 370	12 592	26	1065	75	2087.0	16.6

[1]) excluding reservoirs on the Danube

It was not until 1960 that irrigation was considered in the design of larger reservoirs. By 1970, ten such reservoirs had been built. Those at Vihorlat in east Slovakia ($336 \cdot 10^6$ m^3), Rozkoš near Česká Skalice ($76.15 \cdot 10^6$ m^3) and Nové Mlýny on the river Dyje ($145.3 \cdot 10^6$ m^3) serve mainly for irrigation.

The construction of reservoirs for agriculture is closely linked with the development of agriculture and the available water resources.

Table 1.2. shows the increased demand for water for irrigation and cattle breeding. It is important to remember that irrigation water is irretrievable. Agriculture will constantly continue to need more water and it is presumed that by the year 2000, seventy-one large, and several small, reservoirs will be used for irrigation.

(h) Reservoirs for recreation purposes

Many Czech reservoirs are also used for recreation (Liberec, Bystřička, Vranov, Seč, Kníničky, Orava, Slapy, Lipno, etc.); however, this is not the purpose for which they were originally built. The first impounding reservoir built exclusively for recreation was that on the Botič at Hostivař, built in 1962 and large enough for 36 000 visitors. A recreation reservoir was built in 1973 for the Plzeň region, in České Údolí (Litice) on the river Radbuza.

The reservoirs at Slapy and Orlík give many people the chance to spend their leisure time in a pleasant environment; however, the water in the river Vltava in Prague is now much colder due to the Vltava cascade of reservoirs. Small recreation reservoirs are therefore being built in Prague on small rivulets (Motol, Šárka, etc.).

The main problem with these reservoirs is to ensure the required volume of water, especially during a hot summer and its quality. Reservoirs which supply drinking water must not as a rule be used for recreation.

Table 1.4 Data on reservoir utilization in Czechoslovakia up to the year 2000

Territory	Reservoirs			Purpose of reservoir							
		volume [mil. m^3]			surface water supply for						
	number	storage	total controlled	public water supply	industry	irrigation	other purposes	flood control	hydro-power	recreation	fish farming
Czechoslovakia	310	6128	8466	101	91	71	26	161	75	135	100
Bohemia and Moravia	235	4041	5352	82	62	39	21	103	46	85	53
Slovakia	75	2087	3114	19	29	32	5	58	29	50	47

(i) New reservoirs to be built in Czechoslovakia

With an average annual precipitation of 690 mm, about $89 \cdot 10^9\ m^3$ of water falls on an area of $127\,800\ km^2$ per year. Of this, the runoff amounts to 31%, i.e., $27.7 \cdot 10^9\ m^3$ in an average year. In a low-flow year the runoff is only $16.3 \cdot 10^9\ m^3$. Table 1.3 shows the distribution of the total average annual runoff in the respective watersheds, and the volumes of the 185 reservoirs built up to 1970 and the volumes of the 310 reservoirs planned up to the year 2000.

The total accumulation of water is to increase in Czechoslovakia from 12.9% in 1970 to 22.1% in the year 2000. On a nationwide average this is a large volume, so that it is possible to make good use of the available water.

Table 1.4 gives the data on the reservoirs and their exploitation in Czechoslovakia as given by the Water Management Plan up to the year 2000.

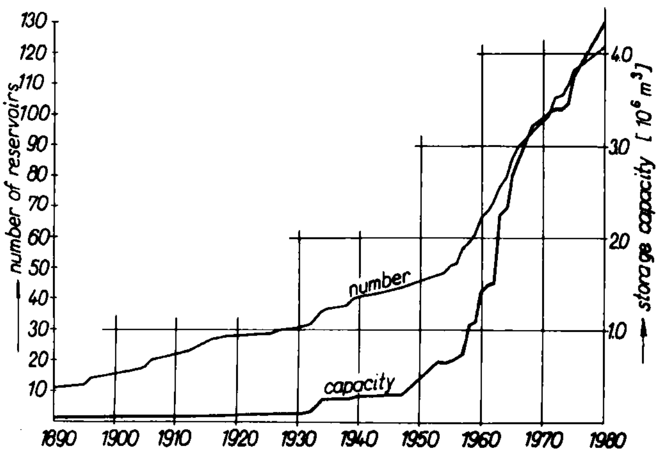


Fig. 1.1 Reservoir construction in Czechoslovakia (World, 1976).

Figure 1.1 shows the number of reservoirs and their volumes included in the World Register of Dams of the International Commission on Large Dams. This list includes only those reservoirs with dams at least 15 m high: the Czechoslovak Water Management Plan therefore has a greater number of reservoirs.

1.2.2 Worldwide construction of reservoirs

Many reservoirs are being built both in developed and developing countries. The greatest pre-war reservoir volume (Hoover $36.7 \cdot 10^9\ m^3$) has already been exceeded by 15 others built during the last 20 years. Table 1.5 lists the 25 largest reservoirs in the world (Mermel, 1975).

Table 1.5 World's largest capacity reservoirs in 1988 (Mermel, 1988)

No.	Name	Country	Year	Volume [10^9 m^3]
1	Owen Falls[1])	Uganda	1954	2 700.000
2	Kariba	Zimbabwe/Zambia	1959	180.600
3	Bratsk	USSR	1964	169.270
4	Aswan (High)	Egypt	1970	168.900
5	Akosombo	Ghana	1965	148.000
6	Daniel Johnson	Canada	1968	141.852
7	Guri (Raúl Leoni)	Venezuela	1986	138.000
8	Krasnoyarsk	USSR	1967	73.300
9	Bennett W. A. C. (Portage Mt.)	Canada	1967	70.309
10	Zeya	USSR	1978	68.400
11	Cabora Bassa	Mozambique	1974	63.000
12	La Grande 2	Canada	1978	61.715
13	Chapetón	Argentina	u.c.	60.600
14	La Grande 3	Canada	1981	60.020
15	Ust-Ilim	USSR	1977	59.300
16	Boguchany	USSR	u.c.	58.200
17	Volga–V. I. Lenin (Kuibyshev)	USSR	1955	58.000
18	Serra da Mesa	Brazil	u.c.	54.000
19	Caniapiscau (KA-3, KA-4, KA-5)	Canada	1981	53.800
20	Upper Wainganga	India	1987	50.700
21	Bukhtarma	USSR	1960	49.800
22	Atatürk	Turkey	u.c.	48.700
23	Irkutsk	USSR	1956	46.000
24	Tucuruí	Brazil	1984	45.800
25	Turukhansk (Lower Tunguska)	USSR	u.c.	45.000

u.c. denotes under construction

[1]) The major part of this reservoir is the natural capacity of a lake

To forecast the future development of water management, not only must the future requirements for water be known, but also the future regime of the water resources, which will continue to be increasingly influenced by human intervention. To make such a forecast is not easy, as

– there is no strict dividing line between the water consumers (people, industry, agriculture) and between the water users (hydro-power, navigation, fish farming, recreation), as, e.g., hydro-power, by its large reservoirs, greatly changes the hydrological regime and the properties of water, and evaporation causes water losses;

– the hydrological regime and the properties of the water are greatly changed by agrotechnical and forestry measures, urbanization, etc., which are neither consumers nor users of water.

Withdrawals of water decrease the flow in rivers and reservoirs influence the water sources. The need for water, as well as its irretrievable consumption, is rapidly growing in all continents. The value of the annual water need (p) on Earth came to totals of 400, 1100 and 2600 km^3 in 1900, 1950 and 1970, and the annual irretrievable consumption (s) to 270, 650 and 1550 km^3, respectively. According to forecasts $p = 6000$ and $s = 3000$ km^3 in the year 2000. In 1970, the withdrawal was divided as follows: public water supply 120, industry 500 and irrigation 1900 km^3. Irretrievable annual consumption for irrigation came to 1300 and evaporation from reservoirs to 70 km^3. The total reservoir volume is 5000 km^3 and the surface area 300 000 km^2 (Mirovoj, 1974).

Large reservoirs are usually multi-purpose reservoirs. The Aswan High Dam has a capacity of about $30 \cdot 10^9$ m^3 of dead storage, $70 \cdot 10^9$ m^3 of active storage for electrical power and irrigation and $30 \cdot 10^9$ m^3 for flood prevention. The complex system in the Snow Mountains of Australia has $2.25 \cdot 10^9$ m^3 water for irrigation and a total output of 2 770 MW from 15 hydro-power plants.

The Bulgarian reservoirs (the largest is Isker—$670 \cdot 10^6$ m^3) store water for irrigation and hydro-power. Typical for Bulgaria are systems where large reservoirs upstream regulate the flow for the hydro-power plants, and the large reservoirs downstream of the last power plant change the flow according to the irrigation needs.

Plans for the construction of reservoirs cannot be made without determining their future tasks in the cascade of reservoirs and in the water management systems (see Chapter B.1). Due to the greater exploitation of water an ever-increasing number of reservoirs are becoming a part of extensive systems. The designers of these systems are faced with qualitatively new and much more difficult tasks than designers of individual reservoirs. Probability and system approaches will become the rule and the application of system sciences and modern computer technology will become a necessity.

1.3 MODERN TRENDS IN RESERVOIR-DESIGN

The problem of water storage in reservoirs was solved according to the function of the reservoir and according to the construction means of the given period. In the course of time new solutions were found and the development of the respective sciences made it possible to perfect the methods. Czechoslovakia stood at the forefront of reservoir development worldwide.

In Czechoslovakia the development of the analysis of water management of reservoirs can be divided into three stages:

(a) the solution of the function of an isolated reservoir with a short-term, or within-year control cycle, by direct methods in real series (up to 1960),

(b) the solution of the function of isolated reservoirs with a within-year and over-year control cycle, using probability methods (since the 1950s),

(c) the solution of the function of reservoirs in single purpose and multi-purpose systems (since 1970).

The design of the reservoirs must take into consideration the relationships between the water resources (hydrological conditions), the need for water (economico-social conditions) and the size of the reservoirs (accumulation conditions).

Hydrological data

Long-term measurement of precipitations and runoffs, processing the results of the measurements, up-to-date hydrological and hydrometeorological information ensure a good knowledge of the hydrological conditions.

On the initiative of J. Stepling, meteorological data began to be recorded in the Prague Klementinum as early as 1752, which makes it the oldest meterological station of the former Austro-Hungarian empire. In Slovakia, the first station was established in 1789 in Kežmarok.

Bohemia was also among the first countries to organize hydrographic services. After the floods of 1872 and after the exceptionally low-water period in 1874, a hydrographical service was established for the Kingdom of Bohemia in 1875.

Thanks to this initiative long series of measurements exist. However, exceptionally high- or low-water periods were recorded even earlier: the great flood in Prague, Ústí on the Labe and Děčín have been measured since about the year 1000, water stages have been measured regularly by the water gauge near the Staroměstský dam in Prague since 1825. After 1851, further water gauges were installed on the Labe in Mělník, Litoměřice, Ústí and Děčín. The oldest water gauges in Slovakia can be found in Bratislava (1823) and Komárno (1830) on the Danube. Hydrological services were introduced in 1886. Since 1919, hydrological services for the whole country have been supervised by the State Hydrological Institute in Prague, which in 1954 merged with the Meteorological Institute to form the Hydrometeorological Institute.

Nine hundred and seventy four water gauges have been installed in Czechoslovakia, thus there is one gauge for every 130 km^2.

The long-term hydrological measurements were processed. After 1960, data from the most important water gauges were processed for the thirty-year period from 1931 to 1960 to serve as a basis for future studies. The result of this work was the study *Hydrological Conditions in the ČSSR (1965–1970)*, the contents of which are unique.

The fifty-year series: 1931 to 1980, has been processed to serve as the hydrological data for future periods. In view of the constantly more progressive non-stationary character of hydrological series, it is rather doubtful whether more recent, and shorter series, can be considered as representative in view of the existence of the longer series which date back a long time.

The Hydrometeorological Institute processes the basic hydrological data in three cycles, with monthly, annual and long-term (10, 20 years, etc.) intervals. Figure 1.2 shows a diagram of a monthly balance (Daňková *et al.*, 1975). It can be seen that this is not only a calculation of the average daily flow, but the optimization of the number of hydrometric measurements, in order to check the accuracy of the stage-discharge

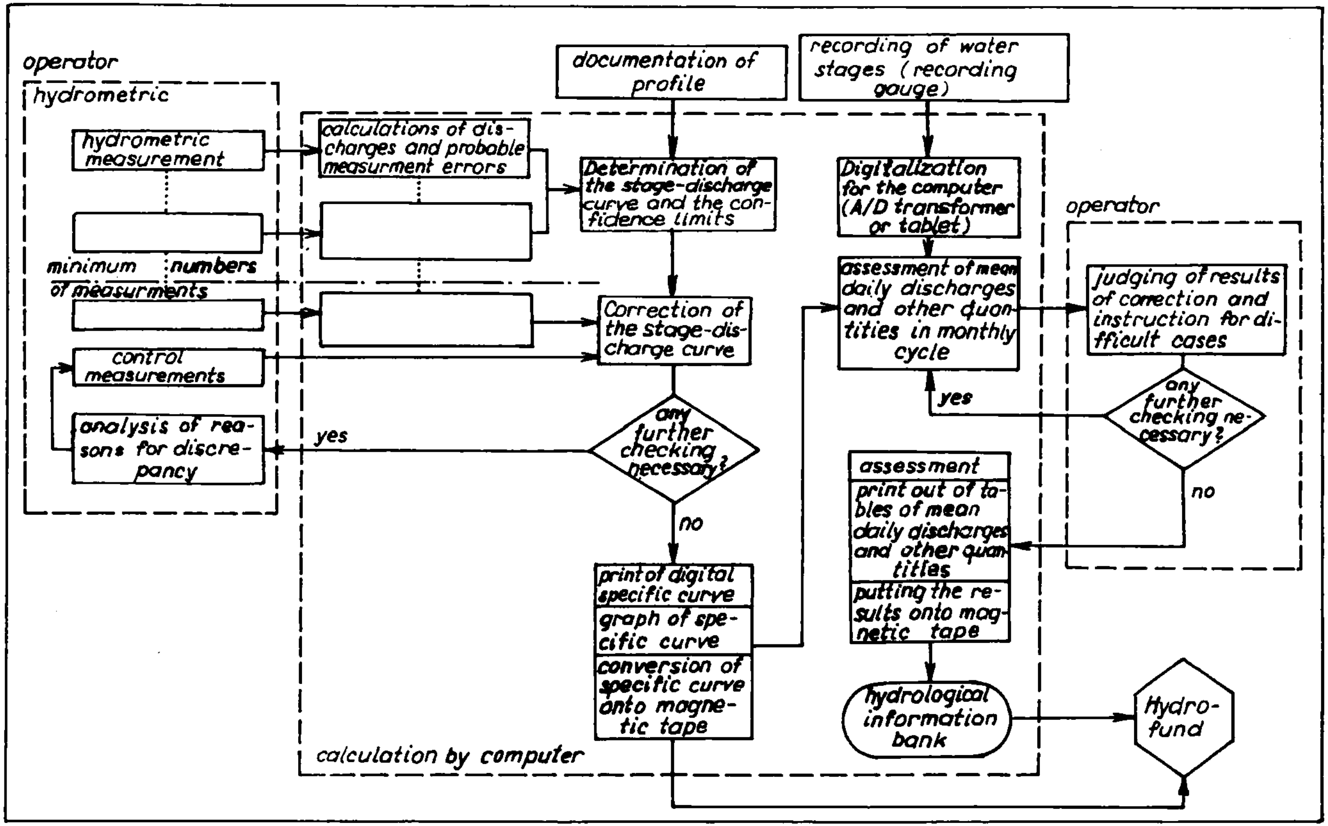


Fig. 1.2 Automation of monthly flow balance.

curve. The diagram also shows the links between the hydrological data bank, the Hydrofund and the calculation process. Data in the bank are processed in annual or long-term cycles.

Hydrology is one of the basic factors of the information system of water management (ISVH), which is part of the national information system on the territory (ISÚ), established in 1970. The basic system of hydrological information consists of information files and sub-files according to the hydrological division of water occurrence in nature; a survey is shown in Table 1.6. The library of the hydrological sub-files programmes is constantly being supplemented.

Table 1.6 Survey of registers and sub-registers on surface water in the hydrological data-bank

Register	Sub-register
A.I. basic register of discharges	II. sub-register of derived water discharge indices
	III. sub-register of extreme discharges phases
B.I. basic register of temperatures	II. subregister of derived water temperatures
C.I. basic register of water stages at selected stations	

The Hydrofund records, registers, documents and balances all activities relating to water management in an attempt to obtain knowledge of, and to assess the natural water fund. The Hydrofund therefore is able to give information about the natural water circulation regime and about the capacity of resources in the respective regions, and it systematically supplements and defines the hydrological balance. The Hydrofund thus helps to determine the water balance, and its control, over the whole country.

To evaluate the natural amount of water, the Hydrofund registers the results of observations and measurements at the gauging stations of surface water and subsurface water, of the precipitation gauges and atmometers of the Hydrometeorological Institute's network, as well as of the stations of other organizations, data on the withdrawal of water and on mineral and mine water. This information is processed by the Hydrofund from various angles, in order to compile balances, hydrological studies, studies of the thermal regime, hydrological manuals and other documents which are indispensable in designing reservoirs.

Hydrological forecasts play a large part in water management in reservoirs. However, in our physico-geographic conditions the hydrometeoric forecasts only have a limited prediction period. It can be increased, and this is especially important during floods, by hydrosynoptic forecasts which use synoptic maps with actual data on the state or the presumed development of the atmosphere (Kakos, 1976). Even though the meteorological forecasts are not yet sufficiently reliable and accurate, the hydrosynoptic forecasts of the Central Forecasting and Water Management Information Service (ÚPVIS) have already provided timely notifications or warnings several times since they started being issued in 1971.

The first water management calculations

Hydrotechnical projects have been designed on the basis of water management calculations since the end of the 19th century. At that time these calculations were only simple empirical calculations, resulting in inaccurate basic parameters, frequently leading to damage or even catastrophes.

The first hydrological study was undertaken in 1893 for the dam on the lower Morava. The first books which reflect the standard of the water management calculations of that time are by Jilek (1904, 1907, 1909) and Stupecký (1909, 1911). Graeff (1873) gives the basic equation of the performance of a reservoir in a differential form

$$F\,\mathrm{d}h = P\,\mathrm{d}t - O\,\mathrm{d}t$$

as well as

$$F\,\Delta h = P\,\Delta t - O\,\Delta t$$

where F is the surface area of the reservoir at the level h;
h – water level in the reservoir.

The analytical solution of the differential equation is derived and an empirical calculation of the flood storage effect of the reservoir is carried out under simplified presumptions.

The graphical solution of the basic equation of reservoir function was introduced in the 1880s. In 1897, Kresnik published a graphical solution of the flood-control effect of a reservoir using a mass curve of the inflow and outflow. Stupecký (1909) published a graphical and numerical (in the form of a table) solution for conservation and flood-control reservoirs. However, it was Ježdík (1936) who systematically elaborated graphical methods in water management.

Development of the calculations of the flood-control effect of reservoirs

Most of the older reservoirs served only for flood protection. The obvious relationship between the level of the reservoir and the outflow from the reservoir was the reason why the flood-control effect of reservoirs was solved earlier than the designs of conservation reservoirs.

The first solution, from which others were derived, was presented by O. Z. Ekdahl in Sweden (1888). This solution does not require the drawing of a mass curve and chooses a time interval Δt, for which the change of the level Δh in the reservoir is found (other authors used the change of the outflow O); the result is the time behaviour of the water level $h = f(t)$.

Thanks to T. Ježdík the most frequently used Ekdahl type of solution in Czechoslovakia is that by Visentini (1932). This calculation requires, besides the given flood $P = f_1(t)$, the construction of two auxiliary curves

$$\frac{O}{2} = f_2(h) \qquad \text{and} \qquad \frac{O}{2} + \frac{V}{\Delta t} = f_3(h)$$

where P is the inflow to the reservoir,
O – the outflow,
t – time,
h – water level,
V – reservoir volume.

The result is the outflow curve

$$O = f_4(t)$$

A step forward was Potapov's (1933) calculation, which was the first to introduce the relation $O = f(V)$, by which he replaced the two relations $O = f'(h)$ and $V = f(h)$, which means that besides the given flood $P = f_1(t)$ only one auxiliary curve $O = f_2(V)$ was necessary; the result of this solution is the time behaviour of the outflow $O = f_3(t)$. Potapov's calculation also used the variable interval Δt for the first time.

In the 1950s, several other new calculations needing only one auxiliary curve were used. These can be found in Mallet-Pacquant's book on earth dams (1951), in the works of Sorensen (1952), Ho-Ci-Huan (1954), Urban (1956) and Záruba (1961).

Záruba's calculation is the simplest. The only auxiliary line is the stage-discharge curve of the outflow device $O = f(h)$. The depth-storage relation is approximated (with sufficient accuracy) by a straight line or by a broken line.

An accurate calculation using the mass curve of the inflow was presented by Kožený (1914), who worked with subsidiary curves $V = f_2(O)$, $O = f_3(h)$, $\sum P = f_1'(t)$; the result of the calculation is the outflow curve $O = f_4(t)$. This calculation differs from the previous ones; here the depth interval of the change of the water level in the reservoir Δh is chosen and the respective time interval Δt is sought. Work with a mass curve takes more time and makes the solution less accurate.

A solution for the regulation of a flood during its passage through the reservoir by operating the gates was presented by A. Bratránek (1939) both for the lowering of the gates up to the spillway crest and for raising them above the crest. This solution is both numerical and graphical and is based on the presumption that the gates move at a constant rate and that the inflow is constant. J. Urban modified the graphical solution for a general rate of inflow and a general rate of the lowering and raising of the gates.

Urban's work (1956) with about twenty-five solutions reflects the great contribution made by Czech engineers in solving the retention effect of reservoirs.

To solve the retention problem successfully, not only the maximum flood discharge must be known, but also its duration, i.e., its volume. For studies of probability and statistics the flood volume is just as important as the maximum peak discharges.

Large over-year reservoirs play an important role in flood control even without limiting the conservation function. An economically justified solution of the flood control effect of a reservoir is no longer based on the design of a control volume for the given normal runoff, but on a probability technico-economical analysis, which takes into account.

– the flood control effect of the active storage capacity of the reservoir,

– the transformation effect of the surcharge capacity on the lowering of the maximum discharges,

– it is only for such a modified set of transformed floods that the actual flood control volume can be designed, taking both economic and other factors into consideration.

As the result of more effective exploitation of water resources, no single-purpose reservoirs have been built in Czechoslovakia for the last 40 years and even the multi-purpose reservoirs have a relatively small flood-control volume.

The development of solutions for conservation reservoirs

Jilek (1909) suggested a solution of the reservoir volume with the help of a direct method expressed in basic curves and mass curves of real hydrological series, both for constant and variable release. In pre-revolutionary Russia, also, the first solutions to the problem of controlled release were based on the balance between the given laws of inflow and release. The first work, published by Masticky (1912), dealt with water used for irrigation.

Ježdík (1936) published a general solution for a conservation reservoir and an analysis of the solution for a balancing reservoir. Ježdík was the first person to publish a complete water management plan for a hydro-power plant with a reservoir. Later a graphical solution was added, which took into consideration the losses caused by evaporation. The advantages of this solution were reflected in the optimum alternative solution and later in the solution with discharge hydrographs, introducing various reliability water supply. These solutions partly lost their importance with the introduction of the probability theory and modern computer methods. However, this solution has retained its significance up to the present for its general character and clear approach. This is proved, e.g., by the fact that random flow series in which flow regime calculations use the same methods as real series are used with increasing frequency.

Water management calculations, however, depend not only on technology, but mainly on the principles of solving the problem of the best utilization of water. Up to the 1930s the problems were simple: to solve, for real time series of hydrologic data and the presumed use of water, the necessary reservoir volume for a guaranteed release, i.e., one hundred per cent security with regard to the given observed hydrological series. This practice was in use up to the 1960s.

These simple problems of release control were overcome in practice by Soviet experts, first of all in hydroenergetics: Morozov (1948), Gubin (1949), Zolotarev (1950) – and not much later also for release control and water management: volumes of the edition AN USSR *Problemy regulirovanija rechnogo stoka*, Kritsky and Menkel (1950, 1952) and others.

Hazen (1914) was the first person to use mathematical statistics for reservoirs calculations. In his work, he derived graphs for the calculation of over-year reservoirs in relation to the mean annual runoff, the variation coefficient and the dependable yield. Hazen used the runoff from fourteen rivers to compile a 300-year series. However, his method did not take root.

Kritsky and Menkel (1932) found a method of determining the over-year reservoir volume for a uniform withdrawal and a given component of reliability. At the same time, they also elaborated a method to determine the seasonal component of the reservoir volume. In 1935 the same authors presented a more perfect method based on an accurate statistical calculation of the outflow. On the basis of this method, Pleshkov (1939) compiled a diagram which greatly facilitated the calculation of the reservoir volumes for $C_s = 2C_v$ for the reliabilites $p = 75, 80, 85, 90, 95$ and 97% and for the coefficient of the yield α up to 0.90.

Ivanov's (1946; Morozov's 1948) calculation method and graphs are based on hydrological series of eighteen rivers (sixteen Soviet rivers plus the Rhine, and the Croton in the USA) with a total duration of 1000 years. This method helps to determine the over-year component of active storage capacity. A computed skewness coefficient C_s (i.e., not only $C_s = 2C_v$) can be used and the α-values reach up to the value $\alpha = 1.0$.

Savarensky's method (1940) makes it possible to determine not only the reliability of the water supply, but also e.g., the distribution of the probability of the filling of the reservoir, the probability and depth of failures, the probability of overflow and the conditions of the functioning of the reservoir during the years of the first filling, etc.

These methods were all based on the white noise random character of the annual discharge values. However, studies of the hydrological series showed that the years were apt to form either low or high flow groups (Yefimovich, 1936). This tendency can be quantitatively expressed by the correlation function of the flow series. Kritsky and Menkel's (1959) method introduced into the calculation of the over-year component of the active reservoir storage capacity the correlation coefficient of the discharge in the successive years; Gugli used this method to construct graphs for the value $r_1 = 0.30$.

Methods were also developed to calculate the seasonal volume of a reservoir.

The problems of reservoir volumes for variable withdrawals during the year, e.g., for irrigation, electrical power, etc., were studied to a lesser extent. L. Votruba solved the problem of the reliability of supply of water, according to the occurrence, the duration and the volume supplied, both for balanced and variable withdrawal (Votruba, 1962; Votruba and Broža, 1963).

The methods described above for the calculation of reservoir volumes are inadequate for some of the more complicated cases of flow regulation.

Soviet authors have worked with exact analytical methods with general statistical characteristics of hydrological data*) which made it impossible to use them for more complex problems. However, even this exact concept of a stochastic approach had to be proved, against the "classical" direct methods. In Czechoslovakia, a decisive stand on the use of direct and indirect methods for the release control by reservoirs was taken at the meeting of the Czechoslovak Scientifico-technical Society in April 1946 (Manual, 1965). It was confirmed that the indirect methods were correct, determining the room for the use of direct methods.

A new qualitative element in the calculation of reservoirs was the nontraditional processing of hydrological data with the help of a mathematical model of a random process, based on a synthetic hydrological series and not on composing real series from several rivers (Hazen, 1914; Ivanov, 1946), nor on any statistical "treatment" (Lyapichov, 1955), but on modelling according to the principles of the theory of stationary stochastic processes (including their correlation functions). To apply these methods it was necessary to determine whether they suited this purpose, and it was also necessary to use modern computer methods without which the calculations could not be made.

An approach, referred to as the Monte Carlo method (random sample, random experiments), made it possible to reach mathematical solutions by multiple random experiments. This method is especially expedient where the algorithm is so complicated that an analytical calculation is practically impossible. In hydrology, it is thus possible to obtain a synthetic series of an arbitrary length, which corresponds to the logic of the modelled process, i.e., the process of the fluctuation of the river runoff.

The first primitive experiment to model hydrological series in this way was made

---

*) The analytical method means that general probability characteristics of release, filling of the reservoir and other variables determining the regime of the given reservoir are constructed according to the general probability characteristics of the flow series and in accordance with the given rules of controlled outflow. This method does not study the respective realization of random processes.

by Sudler (1927). Kritsky, Menkel and Rybkin published their work on the modelling of hydrological series in 1946. From 1954 to 1959 Moran and Gani published studies on the application of the Monte Carlo method in the calculation of reservoirs; management of water in reservoirs was seen by them as one group of a class of probability problems known as "accumulation".

The most complete study of the use of the Monte Carlo method for the calculation of over-year control of the release from reservoirs was presented by Svanidze (1961, 1964) and with application to hydro-energy by Reznikovsky (1969, 1974). Just as in the previous studies, here too, the modelling of the series was based on the principle of the simple Markov chain. The results of these studies were graphs similar to those by Pleshkov and Gugli, however, for a greater number of values of the coefficient of skewness and coefficient of correlation of the release for the previous and following years.

An analysis of the hydrological series, however, showed that their correlation functions have, in most cases, a harmonious character; hydrological series can therefore be realizations of sequences with a periodic component or also of a higher order Markov chain.

From an analysis of the hydrological series, Nacházel (1965, 1975) derived a solution of the over-year component of the active reservoir storage capacity using a composite Markov chain with a damped harmonic correlation function, which is closer to the actual hydrological regime and gives more probable results than the calculation based on the simple Markov chain. In the USSR, Chomeriki (1963, 1964) studied the cyclic fluctuation of river runoff and its influence on the over-year release control.

A drawback of all mentioned probability calculations of the over-year release control was that the reservoir volume was divided into the over-year and the within-year components. The total volume could only be calculated by the balance method with an adequately short time interval. The interval of one month is sufficient for an over-year control. Votruba (Votruba and Broža, 1966) performed such a graphical-numerical solution for a 110-year series, 1851 – 1960, for the Labe in Děčín.

Even a series of such length, however, is too short for reliable over-year control. Therefore, another step forward in the calculation of the over-year storage function of the reservoir was the modelling of synthetic, arbitrarily long series which divided the flow within the year. Two methods of modelling were developed, i.e., the so-called fragment method of Svanidze (1962) and the regression model of flow series (Kos, 1969). The current balancing method is then used for the calculations. Its advantage is that even the most complicated water management problems can be solved in this way. The disadvantage of the large number of calculations have been overcome by the use of computers.

A further development stage in the calculation of the conservation function of reservoirs was the introduction of the non-stationary approach to the processing of hydrological data by stochastic methods. Nacházel and Patera (1975, 1976) did much pioneer work in this field in Czechoslovakia.

As a result of human activities, the non-stationarity of the hydrological regime is going to increase in the future. It is therefore important for the calculation of the conservation function of a reservoir to predict the main characteristics of the future hydrological regime some decades hence, i.e., at a time when the reservoir designed today actually starts to serve its purpose. This forecast should be reflected in the reservoirs being designed today. The trend of research was outlined in the resolution adopted at the Symposium on the Methods of Flow Control through Reservoirs, held at the Technical University in Prague in 1974 (Broža, 1975).

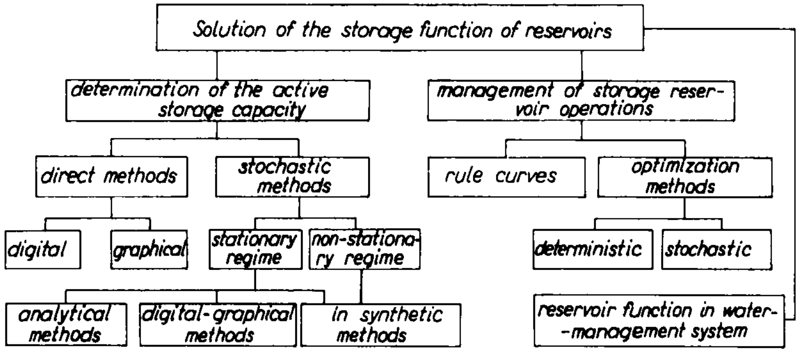


Fig. 1.3 Methods for analysis of the reservoir storage function.

Research into the conservation function of a reservoir should also consider the role of the respective reservoirs in the water-management systems (see Chapter 15). This aspect must be reflected in the design as well as in the control of operations. Figure 1.3 shows a survey of various solutions of the reservoir conservation function.

Direct methods can be considered as closed while the others are still open and will be the subject of further research.

1.4 PERFORMANCE OF THE RESERVOIR

A reservoir, in the broadest sense of the word, is an enclosed volume (vessel) which can be filled and emptied repeatedly over a period of time, with solid, liquid or gaseous substances.

Thus, for example, the term reservoir may be applied to a cement bin which is used to make up the difference in the amount of cement supplied from the cement works and the amount used for construction. A bin of gravel aggregates for concrete has a similar regulatory function in the links between production, transport and utilization of the aggregates. Silos for grain and storehouses for building and other materials have a similar function.

All the above-mentioned reservoirs have one and the same task: to accommodate fluctuations in the supply of substances (inflow) to the reservoir with fluctuations in the withdrawal of substances (outflow) from it; in other words, to permit the proper management of the flow of materials. If the rate of supply were identical to the rate of withdrawal, no reservoir would be necessary. A reservoir becomes necessary only where inflow and withdrawal are temporarily out of balance. The solution principle is therefore essentially the same for all reservoirs; the problem becomes more difficult if the rate of inflow cannot be regulated, as is the case with impounding reservoirs.

Among reservoirs in general may also be included those which do not have a clear release control, because the release is not subject of our interest. The purpose of such reservoirs is to create a particular type of environment: a recreation reservoir, a pond for poultry farming, etc. The water is not accumulated to be used for a specific purpose, but rather to maintain a particular environment.

1.4.1 Basic performance of the water storage reservoir

The basic data for the solution of the behaviour of reservoirs are those concerning the inflow into the reservoir and the demands on the withdrawal, or release from it. Solution of the function of the reservoir is unambiguous if the law of inflow and the law of outflow are determined by the relations

$$P = f_1(t) \tag{1.1}$$

$$O = f_2(t) \tag{1.2}$$

The two relations may be mutually dependent or independent. As a rule, the input and discharge behaviour are so complicated that they cannot be expressed analytically, and therefore tables or graphs (time curves) are used. These may be continuous or discrete, and for the solution of some of the problems their statistical characteristics can be used.

The relationship inflow, outflow and retention of a reservoir is given by the equation

$$P - O = R \tag{1.3}$$

i.e.: inflow − outflow = retention (all in $m^3 s^{-1}$).

If $P > O$, R is positive, the amount of water in the reservoir increases (the reservoir is being filled); if $P < O$, R is negative, the amount of water in the reservoir decreases (the reservoir is being emptied).

Equation (1.3) is the basic equation of reservoir performance; it is very simple, but because of the complicated behaviour in time of the quantities therein, it cannot, as a rule, be solved analytically.

Figure 1.4 shows the general pattern of inflow, P, and outflow, O, and the resulting retention behaviour, R. The value R_{max} is obtained by moving the line O in the direction of the ordinates until it becomes a tangent to the line P. The tangents to the lines P and O at time t_m must therefore be parallel.

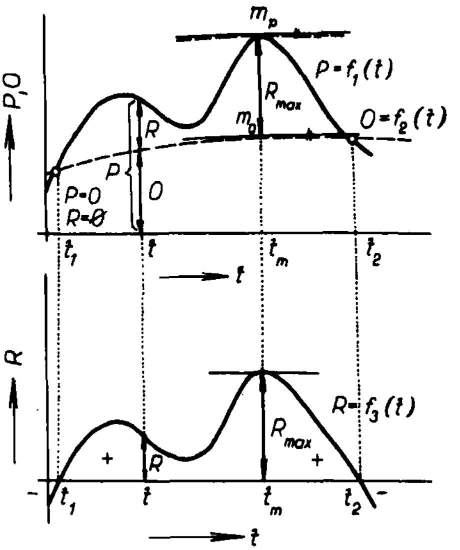


Fig. 1.4 Schematic representation of the basic reservoir function

At times t_1 and t_2, $P = O$, i.e., $R = 0$ and is transitional between positive and negative values. However, points t_1 and t_2 are not the same, since at time t_1 the sign of R changes from negative to positive, and vice-versa at t_2. At time t_1 the reservoir starts to fill and continues to fill until time t_2, when the amount contained reaches its maximum and the reservoir starts to empty.

The basic equation of reservoir performance (1.3) is derived with respect to t:

$$\frac{dP}{dt} - \frac{dO}{dt} = \frac{dR}{dt} \tag{1.4}$$

When $R = R_{max}$ then $dR/dt = 0$ and therefore

$$\frac{dP}{dt} = \frac{dO}{dt} \tag{1.5}$$

Since the terms dR/dt, dP/dt, dO/dt express the gradients of the tangents to the respective curves, we find that in time t_m, when $R = R_{max}$:

(a) the tangent to line R and the axis t are parallel;

(b) the tangents to the lines P and O are parallel.

We usually solve the basic equation of reservoir performance numerically or graphically, by dividing the time, t, into finite intervals, Δt, for which we determine the mean values of P and O, being taken as constant for the period Δt. The precision of the procedure using these discrete values depends on the length of the intervals and is usually not less than the accuracy of the initial data.

1.4.2 Classification of reservoirs according to origin and location

Generally speaking, any enclosed volume with a variable water content has the function of a reservoir. Such a system can, for example, be a lake, a storage reservoir, a pond, a special-purpose reservoir, a water-works reservoir, etc. However, other formations not generally thought of as reservoirs have a similar effect on the runoff from within the watershed: inundations arising from accumulated flood water, depressions in the terrain, the soil, vegetation, snow etc.

Reservoirs are either natural or artificial:

1. A *natural reservoir* is a hollow (a basin) or cavity filled with water which arrives without human intervention. It can be of tectonic, volcanic, glacial or karst origin, etc. To increase its regulatory effect, it may be equipped with a regulating structure at the outlet.

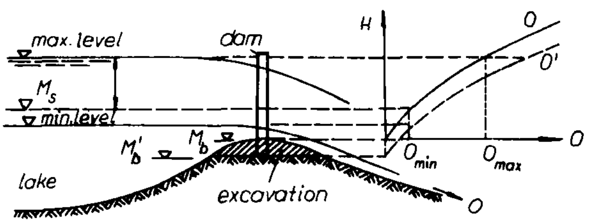


Fig. 1.5 Schematic representation for use of a lake for release control

Figure 1.5 shows a longitudinal section of a lake outlet (lakes being the most common natural reservoirs), and the relationship between outflow, O, and the water level in the lake, H. The height of the dead storage level, M_s, and the active storage between M_s and the maximum level are determined by the condition that the maximum backwater must not be exceeded, and that the minimum regulated flow should be O_{min}.

The active storage can be increased in two ways:

(a) by increasing the maximum backwater above the maximum water level,

(b) by dredging the bottom and the outlet from the lake so as to enable, for M_s, a discharge of O_{min} to occur.

Figure 1.5 shows the required dredging depth and specific curve, O, at which $M_s =$ = min. level.

The depth of the active storage, h_z, of lakes serving to control outflow, is usually small since the large area of the lake in itself ensures a large storage volume. The cascade of reservoirs on the River Ankara is able to control runoff by virtue of a rise in the level of Lake Baikal to 1.46 m above the average level, equivalent to a storage volume of $46 \cdot 10^9$ m^3. As can be seen, a lake makes it possible to gain, at relatively low cost, a large man-made reservoir volume.

2. A *man-made reservoir* is one made specially the purpose of managing natural processes. It may be a *reservoir without through-flow* (*diversion reservoir*), i.e., not forming part of the course of a stream and having its own artificial supply of water, or it may be a *through-flow reservoir*.

Reservoirs without through-flow can be made (a) by excavation of the ground, by erecting a dam enclosing a basin, or most frequently by the combination of both; or (b) as lateral reservoirs.

Through-flow reservoirs are usually built as impounding reservoirs. If it is not the water of the main stream that is impounded, but rather that of a tributary (with the inflow of water from the main stream), then the reservoir is called a *lateral reservoir*.

An *excavated reservoir*, which is usually bordered by dams made from the excavated material, is built where it is most expedient from a functional point of view, and where a cheaper and simpler reservoir cannot be built (e.g., by building a dam across a valley). Today, excavated reservoirs are mainly built as upper reservoirs for pumped-storage power plants, e.g., Štěchovice (1945), Fig. 1.6; Amalienhöhe in the GDR (1963), Čierny Váh (1981) and others.

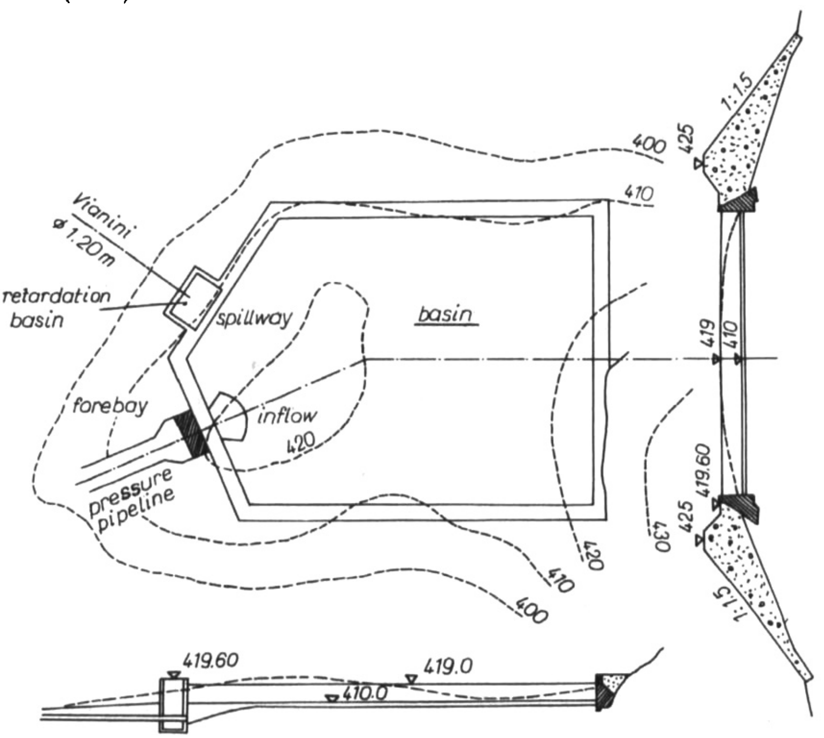


Fig. 1.6 Upper reservoir of a pumped-storage hydro-power plant, Štěchovice (1945).

A *lateral reservoir* is built by dividing and closing off a part of the valley beside the stream by means of a side dam. Such a dam is not a barrage, as it does not dam the valley transversely and does not create an impounding reservoir. For a fairly large lateral reservoir, the average cost of the dam (per unit volume of the reservoir) is usually higher than the cost of that for an impounding reservoir. However, it can still be more advantageous than an impounding reservoir if it helps to prevent the flooding of important installations, housing estates, factories, roads, fertile fields, etc.

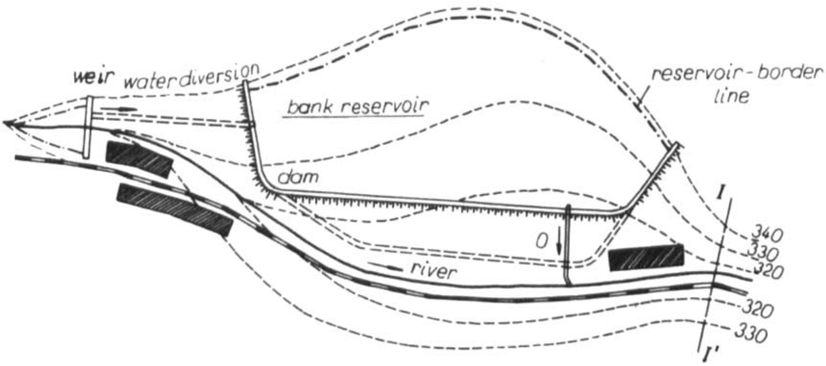


Fig. 1.7 Layout of a lateral reservoir.

Another advantage of lateral reservoirs is that the inflow of water can be stopped at any time, so that it can be protected against floods, polluted water, silting, ice, etc. There are also smaller demands on the outlet and safety devices, and maintenance of the reservoir is reduced. However, compared with the impounding reservoir, the lateral reservoir has a smaller regulatory effect as it can only control that part of the flow in the river which can be brought to the reservoir by diverting structures.

Impounding and lateral reservoirs are compared in Fig. 1.7. If the impounding dam were built in the profile $I - I'$ the reservoir would be larger and would have a relatively short dam, but it would flood a railroad and the buildings in the valley. Expediency suggests shifting the side dam of the lateral reservoir towards the river into the hatched part, a small increase in height and lengthening of the dam greatly increasing the volume of the reservoir.

Lateral reservoirs are *ponds without throughflow*; here the advantages of lateral reservoirs can be exploited fully as the object of the pond is not to control the outflow, but to create an environment for fish farming where perfect control of the inflow to the pond helps to create the best possible conditions for fish farming.

Recreation reservoirs are intended mainly to create a suitable environment for recreation purposes, with clean water, a suitable temperature, an absence of deposits, and no flood problems, etc.; here too, lateral reservoirs prove the most suitable.

A special case of the lateral type of reservoir is the *dry reservoir* (*polder*), i.e., a naturally- or artificially-bounded basin near a stream, which is filled with water only during floods. After the flood recedes it is again completely empty (dry) and can therefore be used for agricultural purposes; it has a retention function, as it reduces the flood discharge in the stream.

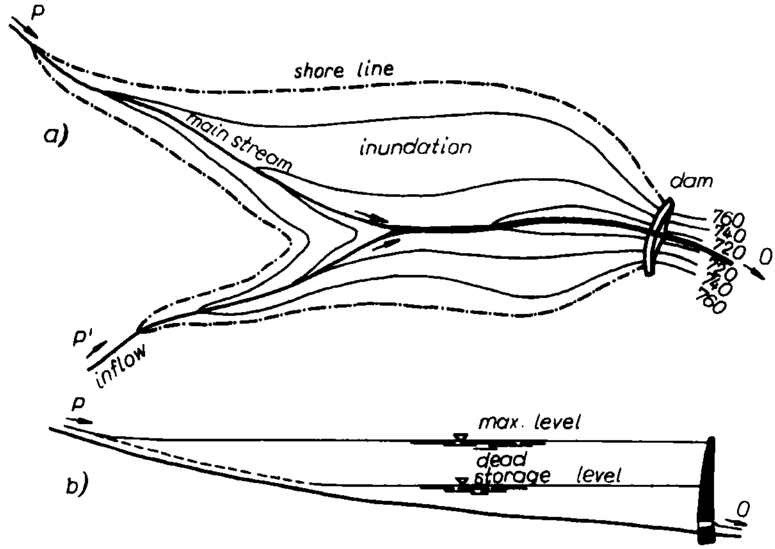


Fig. 1.8 Schematic representation of an impounding reservoir
(*a*) layout; (b) longitudinal section of main stream

An *impounding reservoir* is made by building a dam across the valley of a watercourse, such that the shape of the dam is dictated by the bottom and slopes of the valley, with the upstream face rising up to the backwater elevation (Fig. 1.8). It receives the total discharge of both the main stream, P, and of the tributary, P', and the water leaves it through outlets or spillways discharging on to the riverbed downstream of the dam, or as withdrawal that is taken from the reservoir for any specific purpose. The inflow into the reservoir is given by the total discharge from the watershed to the dam site. For control purposes, the volume contained between the dead storage level and the active-storage level, possibly also the volume of the cavities in the valley rocks between those two levels, is used. The reservoir is filled above the level of active storage only during floods, and thus decreases the flood discharges.

A *tributary lateral reservoir* is an impounding reservoir on a tributary (Fig. 1.9); it is therefore lateral with reference to the main stream. It can control the total discharge of a tributary leading to the dam, and that part of the discharge of the main stream which is diverted to the reservoir. Figure 1.9 shows three ways of diverting water from the main stream to the lateral reservoir.

(a) A water diversion device is built on the main stream and from there the water is led to the lateral reservoir on the tributary. To obtain a natural flow, the diversion facility must be situated above the highest operational backwater in the lateral reservoir. Part of the supply conduit is usually a tunnel, as it is necessary for the flow to traverse the watershed divide between the main stream and the tributary.

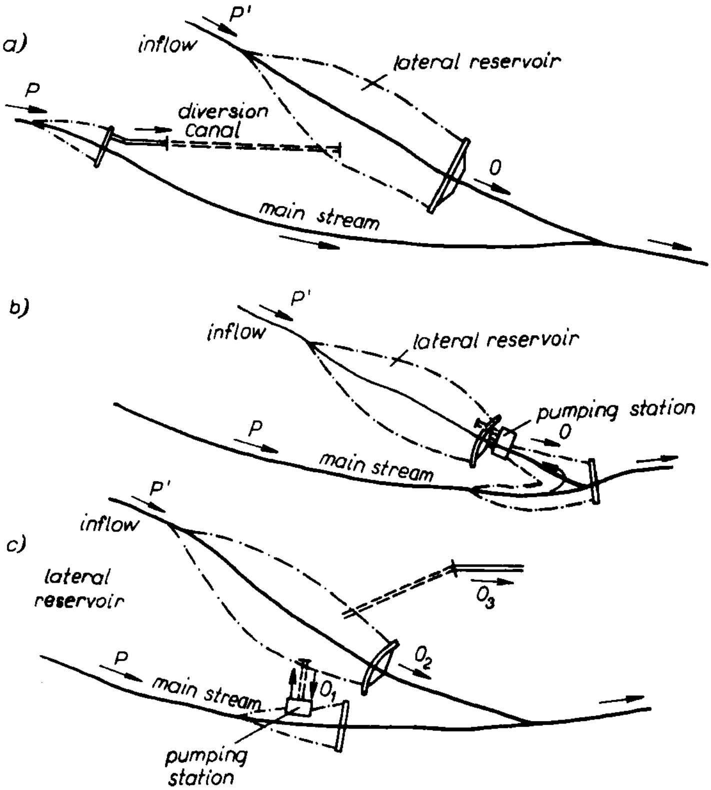


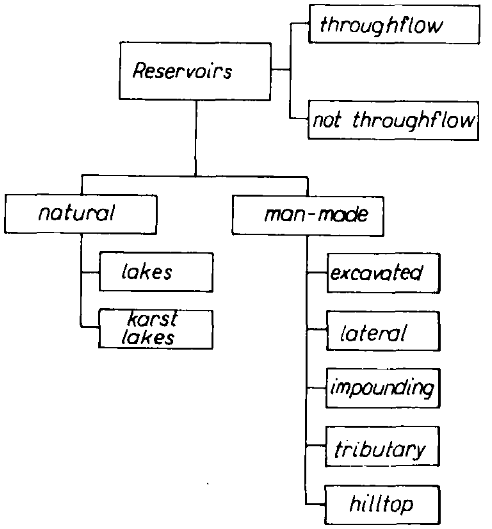
Fig. 1.9 Layout of a lateral reservoir
(*a*) with diversion canal; (*b*) with a pumping station downstream of the dam; (*c*) with a pumping station on the main stream

(b) Downstream of the confluence of the main stream and the tributary, an impounding structure is built so that the backwater reaches up to the dam of the lateral reservoir. Here a pumping station is built which pumps the water from the main stream to the lateral reservoir. To cover part of the power needed, the power of the water flowing back from the reservoir can be used.

(c) The pumping station is built on the main stream in such a way as to make the length of the supply conduit to the lateral reservoir as short as possible. The water from the reservoir can be returned in the same way (O_1 – arrangement of the pumping station as in case (b)), or it can be discharged into the tributary (O_2) if it is used downstream from the confluence, or it can be diverted for use within another watershed (O_3). In the latter two cases, the pumping station houses only pumping machinery.

Discharge control by means of a lateral reservoir may be expedient where there are particularly suitable topographical or geological conditions for the building of the reservoir on a tributary, i.e. where it is not possible to build a reservoir on the main stream, where a reservoir on the main stream might inundate valuable land and structures, where a reservoir on the main stream would require very high capital investment, etc. In all designs for lateral reservoirs, one of the important tasks is to determine the economically optimal design discharge for the supply or pumping equipment for the reservoir.

A *hilltop reservoir* is built near the headwaters of a river or on the watershed divide between two or more rivers, if it is desirable to transfer water from one catchment area to another.



◀ Fig. 1.10 Classification of reservoirs as to their type and place

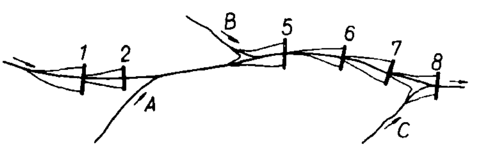


Fig. 1.11 Schematic representation of a cascade (series) of impounding reservoirs

Figure 1.10 gives a diagrammatic classification of reservoirs according to their origin and location.

A *cascade of impounding reservoirs* is a series of functionally integrated impounding reservoirs on the same stream. Figure 1.11 is a diagram of an as yet incomplete cascade of eight reservoirs. A connected cascade of reservoirs is considered mainly where the river water power is to be used in a cascade of reservoirs with hydropower plants. The segmentation of the stream by dams, the determination of the backwater level and the degrees of utilization of the respective reservoirs, are functions not only of the volumes of water and withdrawals, but also of the head. The analysis of a cascade of reservoirs and its operation is one of the most complex problems of water management and hydro-power engineering.

Examples of cascades of reservoirs are those on the rivers Vltava, Volga, Dnepr, Ankara and others.

A *system of impounding reservoirs* is a group of reservoirs on different streams comprising a main stream and its tributaries, the reservoirs cooperating mutually (Fig. 1.12). Operation of the reservoirs must be coordinated if they are power reservoirs and the power plants are supplying power to a combined power network. All or some of the reservoirs can also function co-operatively to protect the downstream part of the valley from floods, and control the flow regime in the downstream river reaches (e.g., the cooperative action of the Vltava reservoirs and the proposed Křivoklát reservoir on the river Berounka, and that of the Váh reservoirs with the Orava reservoir). The cooperation of reservoirs in generating power is not limited to those within one and the same watershed, as the high-voltage network links power plants in different watersheds. The cooperation of reservoirs with respect to water management used to be limited by the boundaries of the watersheds in which they were situated; today, significant diversions from one watershed to another are no longer rare.

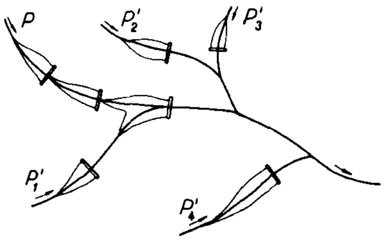


Fig. 1.12 Schematic representation of a system of impounding reservoirs

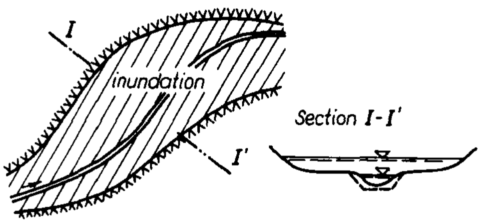


Fig. 1.13 Schematic representation of water accumulation in flood plains

Underground reservoirs are created by damming the permeable soil in a valley with a watertight barrier (underground dam). To let the water infiltrate more easily and more rapidly, it is usually diverted from the watercourse during high discharges by means of deep infiltrations ditches to the sides. The water is then collected from the underground reserves for irrigation purposes, etc. Underground reservoirs require particularly favourable conditions, with respect to the width, depth and properties of the permeable sediment deposits, economic utilization of the valley, etc., if they are to be of any importance for water management.

Accumulation in flood plains (Fig. 1.13) is similar in effect to that of the flood surcharge storage of reservoirs, in decreasing flood discharges. As the area of inundation is usually large, the retention effect of the inundation is also quite significant. If inundation is prevented (by enlarging and cleaning the channel, etc.), the retention effect is also removed and discharge conditions in the lower reaches deteriorate. It can, therefore, be dangerous to proceed with river training which ensures the full discharge of floods from upstream. The mathematical analysis of the retention effect of inundation is, with the exception of the gradient of the water level, similar to that for reservoirs.

Water storage within the watershed greatly influences the discharge regime of the stream. Water is stored

- in cavities and on uneven terrain,
- in the vegetation cover (these two effects influence the runoff mainly during periods of precipitation),
- in the soil as soil and ground water (stabilizing the rate of runoff),
- in the snow cover; a supply of water is created which can dangerously increase spring floods, or can itself cause floods during rapid thawing; on the other hand, the retardation of the infiltration of water into the soil up to the spring is, from an agricultural point of view, a favourable phenomenon; also favourable is the thawing of glaciers and high-mountain snows in the warmer weather, as this helps to raise the low summer flows.
- in ice cover (this decreases the flow in the winter months).

1.4.3 Classification of reservoirs according to purpose

The purpose of a reservoir is to regulate (control) discharges, i.e., to adjust the natural behaviour of the flow in the stream according to the requirements of various water-management schemes.

According to the basic purpose, reservoirs are divided into:

- those which ensure withdrawals of water, or yield (conservation reservoirs),
- those which lower flood discharges (flood control reservoirs),
- those which create an aquatic environment,
- those which are used to adjust the quality of the water,
- those which trap bedload and wastes.

Frequently, one reservoir fulfils several functions (e.g., water supply and flood protection), and is then referred to as *multi-purpose*. A multi-purpose reservoir may also be one which is used in various ways (e.g., a multi-purpose conservation reservoir may have withdrawal for water supplies, irrigation and hydro-power).

Smaller reservoirs serving local requirements (fire protection, small-scale withdrawal, irrigation, watering of cattle, etc.) are also usually of the multi-purpose type.

Figure 1.14 shows a detailed diagram of the classification of reservoirs according to purpose.

A *conservation reservoir* serves to store water, obtained during periods of excess, for use during periods of shortage. If the inflow exceeds the water demand, the reservoir begins to fill; if the demand exceeds the inflow, it empties. The result is that the required water supply is ensured or that low flows are augmented, i.e., $O > P_{min}$

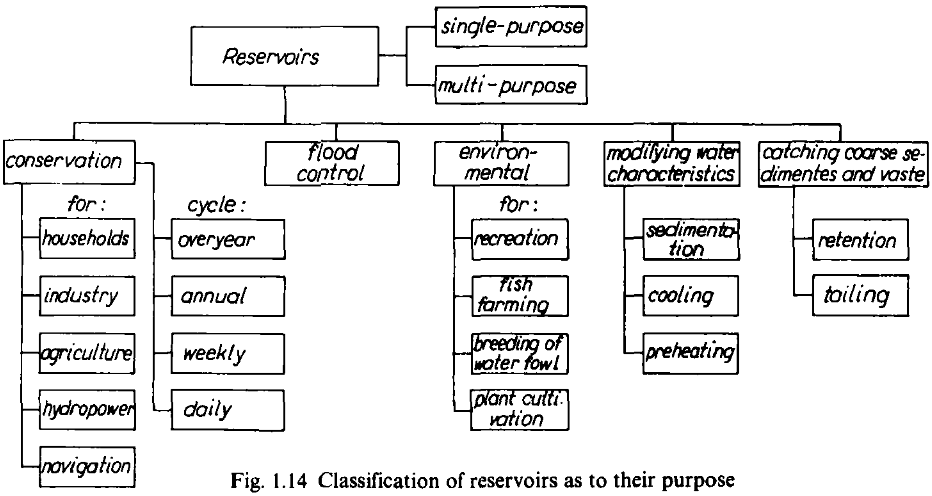


Fig. 1.14 Classification of reservoirs as to their purpose

(Fig. 1.15a). As far as possible, the reservoir is kept full so as to provide a sufficient supply of water in the low-flow period.

The *flood-control reservoir* catches the peaks of flood discharges and thus protects the land downstream from flooding. If the flow exceeds the non-damaging value, O_{nd}, the reservoir fills and the result is a decrease in the maximum flow, i.e. $O < P_{max}$ (Fig. 1.15b). After the flood wave, it is emptied as quickly as possible ($O = O_{nd}$) to be ready to catch the next flood peak.

A reservoir can fulfil both a storage and a flood protection function; it then has an active storage and a flood-control storage. However, the active storage can also fulfil a flood-control function, provided that this volume is at least partially emptied before a flood wave arrives. In particular, the larger over-year storage reservoirs, also lower the flood discharges on account of their supply function, i.e., the intake of water to replenish the storage volume. They also decrease the frequency of floods.

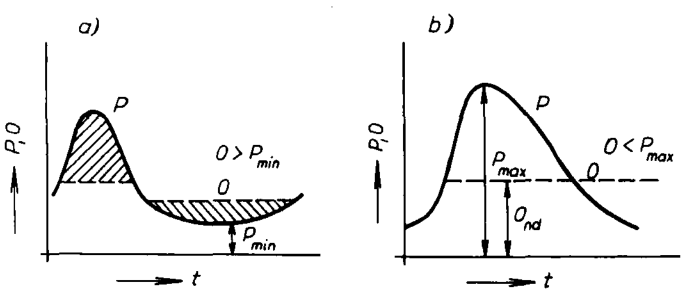


Fig. 1.15 Schematic representation of water supply and flood control functions of a reservoir
(*a*) water supply; (*b*) flood control

According to the duration of the discharge control cycle, basic *control methods* may be divided into:

– *over-year* – the control is longer than one year,

– *within-year* (*seasonal*) – the control cycle is completed within a single water-management year,

– *weekly* – the control cycle is completed in one week,

– *daily* – the control cycle lasts 24 hours.

Figure 1.16 shows the function of reservoirs with various cycle durations in terms of inflow, outflow and water level fluctuation curves.

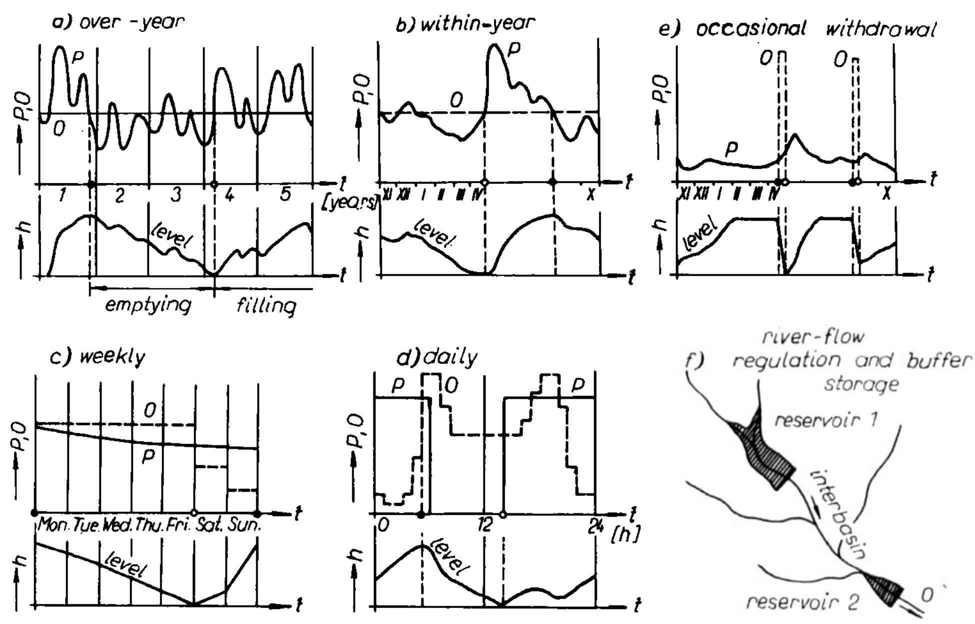


Fig. 1.16 Schematic representation of reservoir function with different cycles

The cycle of the reservoirs is a time interval in a recurring sequence of operations during which the storage volume is filled and the accumulated water used, the space again being completely (or partly) emptied. If we classify reservoirs according to the duration of the cycle, we have to consider the conditions which will determine the size of the storage volume; reservoirs with long cycles (e.g. over-year) always also have shorter superimposed cycles (seasonal, weekly, daily, etc.).

Some reservoirs do not have a regular cycle of filling and emptying, but rather provide for:

– *occasional withdrawals for emergency use* (during breakdowns, etc.)

– *sudden withdrawals* – e.g., for the finer control or river flow regulation (buffer reservoir).

Besides the above-mentioned basic methods of withdrawal control there are some more complicated methods, amongst which we may consider:

– *river flow regulation* – with reaches between the reservoir and the place of withdrawal,

– *in cascades of reservoirs,*

– *in systems of reservoirs.*

Over-year control of the release from a reservoir (Fig. 1.16b) involves a filling and emptying cycle that lasts longer than one year; the annual water demand may be greater than some of the annual discharge amounts, and therefore the excess water from wet years must be used.

A reservoir with an over-year control cycle therefore tends to equalize the discharges of consecutive years, and approximates the total annual release, $\sum O_r$, from the reservoir to the long-term mean annual discharge, $\sum Q_a$, in the river.

The *within-year control* of release from a reservoir (Fig. 1.16b) involves an explicit annual cycle of filling and emptying; the water demand does not exceed the discharge of the current water-management year. Annual control is therefore less demanding with respect to the size of the reservoir volume than over-year control.

The *coefficient of the yield*, α, is defined by the relationship between the required withdrawal, O_p, and the long-term mean flow, Q_a, i.e.

$$\alpha = \frac{O_p}{Q_a} \tag{1.6}$$

It follows that with control aimed at uniform withdrawal, the value of α can be $\alpha \leqq 1$, and therefore $O_p \leqq Q_a$. The value of α will increase with greater evenness of the flow in the river during the year, and with increasing volume of the reservoir.

Seasonal, or *incomplete annual control* of the flow is a particular type of within-year control in which the reservoir functions only for a limited period of the year which may be related to vegetation growth, navigation requirements, the winter season, etc. The control is incomplete when the volume is not large enough to ensure complete control.

Weekly control of the flow (Fig. 1.16c) presupposes smaller withdrawals on some days of the week, e.g., non-working days; there are also usually quite considerable variations during the day. The reservoir is filled, for example, on Saturday and Sunday and is gradually emptied from Monday to Friday, so that one cycle of filling and emptying the reservoir takes a week. The specific characteristics of the weekly cycle depend on the composition of the withdrawal diagram; continuous operations wipe out the differences between working days and rest days.

Daily release control ensures that the difference between inflow and withdrawal is accommodated during the day; one cycle of filling and emptying takes one day (24 hours). Figure 1.16d gives a general example of the variation of inflow, P, and outflow, O, during the day; the inflow to the reservoir is given by the constant pumping rate, P, for a period of 16 hours, and the withdrawal, O, by the hourly averages of water consumption. A common case of a daily control is the matching of a varying inflow with a constant outflow (Fig. 1.17a), or the distribution of a constant inflow into fluctuating withdrawal (Fig. 1.17b).

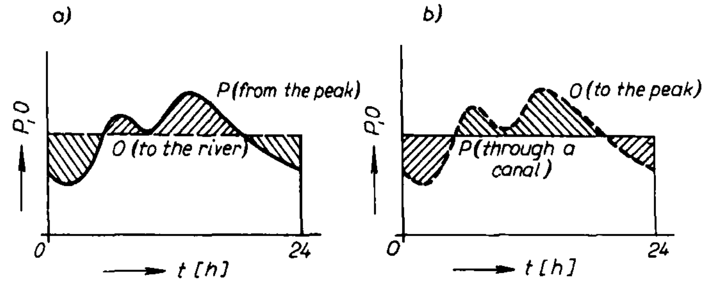


Fig. 1.17 Reservoir functioning with a daily cycle
(*a*) balancing; (*b*) distributing

Storage or *conservation reservoirs* providing for particular types of discharge control are named according to the type of control afforded. We therefore have reservoirs with over-year release control, those with within-year release control, etc., or more briefly, over-year, within-year, etc., reservoirs.

We also use the terms *balancing reservoir* and *distributing reservoir* to refer to two kinds of storage reservoir with daily or weekly discharge control. The equalizing reservoir (Fig. 1.17a) accommodates the differences between a non-uniform inflow and a uniform outflow during the day; equalizing reservoirs are frequently located down-stream of peak hydro-power plants. The distribution reservoir (Fig. 1.17b) has a function opposite to that of the equalizing reservoir; it converts a uniform flow into the reservoir into a non-uniform outflow from it. In plotting the performance of the two types, it therefore suffices to transpose the inflow and outflow lines (Fig. 17b).

Discharge control with a cycle of up to few days (daily, weekly) is termed *short-term release control.*

A control facility allowing for *occasional withdrawal* keeps a certain amount of water in the reservoir for any need that might arise at any time. Such a reservoir might be a fire-protection tank, a locally managed reservoir with a water supply for fire protection, or an emergency store of water for use in the event of failure of a power system. An occasional outflow wave from the reservoir to the river may be allowed to occur for the purpose of transporting wood, for water sports, for flushing the stream, manipulating ice, etc.

When the distance of the intake from the stream is so far from the main reservoir that the outflow from the main reservoir (*1*) cannot be accurately adjusted to provide the required withdrawal, especially if the outflow varies, then the *control of sudden* *discharges* is carried out in a smaller buffer reservoir (*2*, Fig. 1.16f) at the intake point, as a supplement to the main river regulation system. Reservoir *1* takes care of general discharge control, and reservoir *2* provides for finer discharge adjustment to the withdrawal requirements; in other words, the reservoir accommodates the differences between the outflow, caused by withdrawal variations (shocks), and the manipulation of reservoir *1*.

The *river flow regulation* increases the reliability of the yield in the river profile at a greater distance downstream of the reservoir, compared with natural (uncontrolled) discharge from the interbasin. According to the diagram in Fig. 1.16f, reservoir *1* performs the function of river flow regulation, as only that amount of water is released which supplements (compensates) the insufficient inflow from the interbasin to bring it up to the needed withdrawal, *O*.

If there is approximate concurrence of high flow or low flow conditions both within the watershed of the reservoir and in the interbasin, the reservoir will experience difficult operational conditions: in the high flow period the outflow from the reservoir will need to be small or zero, and conversely, in the low flow period the outflow from the reservoir will need to be great. Such conditions place greater demands on the reservoir volume, compared with discharge control for withdrawal just downstream of the dam.

However, for withdrawal at some distance downstream of the reservoir (i.e., with an interbasin area), river flow regulation offers, under some conditions, a higher safe yield than that obtainable without river flow regulation; i.e., if the problem is solved by locating the withdrawal point just downstream of the dam, and to this withdrawal the natural discharges from the interbasin are added.

Discharge control in a cascade of reservoirs is more complicated than that provided by a single reservoir, as the upstream reservoirs affect the pattern of inflow to all the downstream reservoirs. A reverse influence of the downstream reservoirs on the upstream reservoirs can also be observed, if their optimal operation is solved as an integrated operational entity. Such a solution is essential for the hydro-power exploitation of reservoirs, and is relevant also to the use of multi-purpose reservoirs for supply and flood protection.

The *control of release of water in a system of reservoirs* is the most complex problem of all in the control of discharge. The task required cannot be solved without optimizing the structure and behaviour of the system. Technico-economico-social optimization is made the more difficult by the fact that the system is usually a multi-purpose one with conflicting demands on the water, with stochastic inputs, with a dynamic pattern of behaviour, and with important intangible functions (see Chapter 15).

Different types of release control place different demands on the volume of the reservoir. For a stream of a given size and discharge variability, it must generally be larger for longer cycle of release control.

Where long-term release control is practised on a reservoir, there is usually also some short-term control, i.e., in an over-year reservoir, annual, weekly and daily control also takes place. If a solution is sought for over-year control, it does not suffice to consider the mean annual values; the pattern of release and withdrawal within the year must also be taken into account.

The survey in Fig. 1.14 also includes those types of reservoirs in which water is not managed in the proper sense of the word.

Sludge-settling ponds (*tailing dams*) are also mentioned as areas that are naturally or artificially bounded, serving as permanent or temporary storage places for sludge and waste that is transported mainly hydraulically. Although these are not reservoirs in the sense of being within the context of water management, sludge-settling ponds have to be approved by the water management authorities. Sludge-settling ponds by their number, size, and prospective further growth, present a serious technico-economic problem and have a great impact on the environment, necessarily involving water management authorities in the design and operation of sludge-settling systems.

1.4.4 Influences affecting the performance of the reservoir (flow regime, human intervention, complex utilization)

(a) Influence of the flow regime on the reservoir performance

The performance of a reservoir is mainly influenced by the changes of the flow in the river, which depends on weather conditions and on how the river receives its water. The flow in the river Q is derived from a genetic equation (Ogievski, 1952):

$$Q = Q_s + Q_r + Q_g + Q_v + Q_d \tag{1.7}$$

where Q_s is the surface runoff from snow water,
Q_r – surface runoff from rainfall,
Q_g – surface runoff from glaciers,
Q_v – groundwater affluent from valley alluvium,
Q_d – groundwater affluent from the deep aquifer of the catchment.

The terms of the equation have different meanings for rivers of different types and change in time; some can equal zero. The water supply from groundwater can amount to more than half of the annual runoff (rivers in the plains with much alluvia), however, it can also equal zero (in small streams with an impermeable drainage basin). The valley alluvium helps to balance the seasonal runoff in the river and the value Q_v can even be a negative value if the river water level is higher than the surrounding groundwater level. The inflow of groundwater from deep layers changes more slowly and is actually an over-year natural regulator.

Depending on the dominant member in the genetic equation, Lvovich (Dub, 1957) defined four basic types of river regimes: rainfall (R), snow (S), glacier (G) and groundwater (U). He then further divided every type into three groups according to the share of the basic type of supply in the total runoff. He also took into account whether the greatest runoffs occur in spring, summer, autumn or winter. The rivers of Czechoslovakia, with the exception of the Danube, belong to the so-called Oder type, where the rivers receive most of their water from rainfall and an increased rate of streamflow occurs in the spring.

According to their hydrographs, the various types of rivers need various reservoirs volumes to ensure the same safe withdrawal. Let us presume, e.g., an annual control of the outflow for a constant release. Then the hydrograph with one very short high-flow period will require largest reservoir volume.

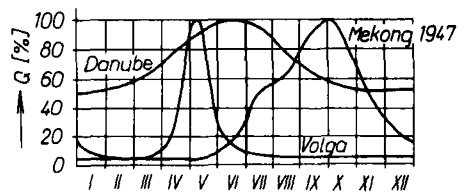


Fig. 1.18 Annual relative discharge curves on the Danube, Volga and Mekong rivers

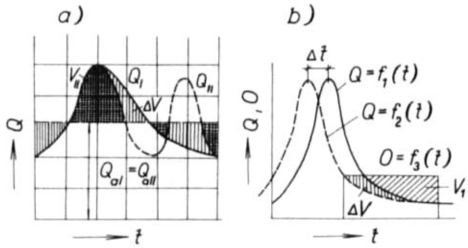


Fig. 1.19 Relationship between reservoir storage capacity and type of river
(a) storage capacity for the 100% relative yield for two types of rivers; (b) influence of the shift in hydrograph on the reservoir storage capacity

Figure 1.18 gives the annual hydrographs for the Danube at Bratislava, the Volga at Gorky and the Mekong at Phnom-Penh. Each of those rivers belongs to a different type: the Danube reflects the glacier character with the greatest runoffs in June, the Volga has the greatest runoff in April and May after the snow thaw and the Mekong belongs to the monsoon region with the highest flow in September and October. What all three rivers have in common is one main wave of increased discharge during the year, which requires the reservoirs to have a large volume. However, the Danube reaches the same yield coefficient $\alpha = O_p/Q_a$ with a smaller relative volume β, as it has a more favourable ratio $Q_{min} : Q_{max}$ and a longer period of increased discharge. For the Volga and the Mekong the ratio $Q_{min} : Q_{max}$ is roughly the same; however, the Volga will require a larger reservoir relative volume β, as the period of increased discharge is much shorter than that of the Mekong.

Rivers with two or more periods of increased discharges during one year generally require smaller reservoir volumes than rivers with a single high-flow period. The diagram in Fig. 1.19a shows the complete balancing of the discharge of the two types of discharge curves. The curve Q_{I} with one wave requires a volume bigger by ΔV than the curve Q_{II} with two waves, for which a volume V_{II} is sufficient.

Figure 1.19b shows the required withdrawal O and two discharge curves Q which have the same shape and differ only in time Δt. The full curve $Q = f_1(t)$ must have a reservoir volume V_1, the dotted curve $Q = f_2(t)$ a volume bigger by ΔV.

However, rivers with one high-flow period have, as a rule, a more regular discharge during the year and the reservoir volume can be designed with a greater accuracy. Rivers fed by rain which can occur at any time during the year, and rivers where snow can start to thaw any time during the winter, have hydrographs with a very accidental character. This fact makes it more difficult to design a reservoir and also to operate it.

The rivers of Czechoslovakia are among those with difficult discharge conditions. If the river regime is stochastic, forecasts for at least the medium-term are of great importance. Attempts at long-term forecasts have so far been imperfect.

The design of reservoirs and the directives for their operation are therefore based on past observations. The information gained from actually observed data is increased by methods of mathematical statistics.

However, not even the law of release can be predicted with a sufficient certainty. Water demand will depend on the development of the economy and forecasts about the future demand of water cannot be any more accurate than forecasts of economic and social development.

Useful for the conditions in Czechoslovakia are the *medium-term (seasonal) forecasts* for at least several weeks or months which

- forecast the discharge according to the recession curve in the dry period,
- forecast of the spring runoff from snow thaw.

Even though the *forecast of the spring runoff* does not concern such a large part of the whole annual runoff as it does for the rivers in the plains of the countries of the north, it is of importance for the management of water in reservoirs at the end of the winter period, which tends to be critical especially for the hydro-power reservoirs. In the slightly sub-normal year, 1935, the approximate runoff from snow was $82 \cdot 10^6$ m^3 in the catchment of the Lipno reservoir on the Vltava, which is more than 40% of the active storage capacity. The methods and results of these forecasts can be found in the works by Votruba, 1949; Martinec, 1961, 1963; Dub and Němec, 1969. Although the general equation for the volume of the spring runoff from snow V_{sn} has about 10 variables, a linear equation proved to be suitable for conditions in Czechoslovakia

$$V_{\mathrm{sn}} = aH_{\mathrm{sn}} + c \tag{1.8}$$

where H_{sn} is the amount of snow storage,

a – a constant,

c – a coefficient.

For the decreased flow from groundwater sources in a rainless period, the following equation is applied for the flow Q

$$Q = Q_0 \, \mathrm{e}^{-bt^n} \tag{1.9}$$

which for some of the Czechoslovak rivers takes the form of

$$Q = Q_0 \, \mathrm{e}^{-b\sqrt{t}} \tag{1.10}$$

where Q_0 is the initial flow,
b – depletion coefficient (natural logarithm of the recession constant),
t – time (in days) from the initial flow to flow Q.
The method of calculation is given in the literature (Balco, 1958, and others).

Rivers in which the flow is regulated by lakes, ponds, inundation, etc., usually need only a small reservoir volume.

(b) Influence of human intervention on the water course regime

Today there is no longer any large catchment that has not been changed by human intervention. Agricultural measures, changes in the composition of plants, decay of forests, building of housing estates, mining, river training, etc., cause a change in the surface runoff which is reflected in the design and operation of reservoirs In processing hydrological data the development in time of the catchment and the runoff from it must be taken into consideration. If two sections of the hydrological series have different conditions, then the series cannot be considered to be homogeneous and cannot be considered as a set of data of the same importance.

If the conditions in the catchment changed continuously during the observation period, it would be possible to use mathematical processing of the runoff characteristics on the principle of observations of unequal weight as in geodesy, etc. (Votruba and Broža, 1966). These calculations, however, would not reflect the changes in nature.

It is therefore necessary to determine, by an analysis of the hydrological series, whether it is non-stationary and then to assess the influence of the non-stationarity on the water-management solution.

It is also necessary to bear in mind any essential intervention in the flow, e.g., the construction of a reservoir or the diversion of part of the flow to another catchment. If we want to prolong the previous "natural" series after the intervention, we must change the measured flow by the influence of intervention. The corrections are introduced in the hydrological data and the corrected data are then used in the same way as the observed data of the same weight.

The transfer of the influenced flow series to the natural series is a demanding problem as the size of the influence can usually be quantified only inaccurately and sometimes there are no observations.

However, in designing the water-management measures, the future hydrological and discharge conditions, as they will exist during the operation of the designed structures, must be considered. Long-term forecasts should therefore be elaborated as they give a more pertinent hydrological basis than any statistical processing of data measured in the past or present. These forecasts are especially important where human intervention has greatly changed the flow.

(c) Influence of the comprehensive utilization of a reservoir on its performance

The respective tasks of a reservoir can be competitive. Table 1.7 shows the matrix of competition of five tasks of a reservoir with four types of competition.

The greatest competition is in the required reservoir volume (1) and the operation (4). In the demand for the amount of water (2) the competition is only between the public supply of water and irrigation; in the demand on the quality of water (3) only between the supply of water (drinking water) and recreation. Completely without competition are recreation and flood control.

The aims of the design and the operation of a reservoir are to solve all these contradictions to ensure a technico-economico-social optimum.

The construction of reservoirs may also have unfavourable consequences:

(a) fertile plots, housing estates and buildings become inundated,

(b) communication between the two banks is hampered,

(c) during fluctuations of the water level in a reservoir mud deposits and banks are eroded with aesthetic and environmental impacts,

(d) waves and ice can cause erosion of the banks,

Table 1.7 Competition matrix of reservoir purposes

Purpose	1. (PWS)	2. (HP)	3. (I)	4. (R)	5. (FC)
1. public water supply (PWS)	–	1, 4	1, 2	3, 4	1
2. hydro-power (HP)	1, 4	–	1, 4	4	1
3. irrigation (I)	1, 2	1, 4	–	4	1
4. recreation (R)	3, 4	4	4	–	
5. flood control (FC)	1	1	1		–

1 – competition as to the volume demand
2 – competition as to the demand on the amount of water
3 – competition as to the demand on the water quality
4 – competition as to the manipulation demands

(e) the water quality can deteriorate: the impact of great drops in the temperature of the water on recreation and irrigation, and the anaerobic processes in the reservoir, etc.

Consequences (a) and (b) are included in the assessment of the capital investment, (c) to (e) can be eliminated by suitable design.

The functions of a reservoir should and sometimes even must be combined with other hydraulic structures.

The comprehensive utilization of water sources usually gives the most satisfactory results as to the need of water and also as to the best economic effect. A set of various purposes of water-management is called a *water-management complex* and the respective purposes are its components. Schemes or structures with several purposes are called *multi-purpose structures.* Water reservoirs are, by their very nature, the most complex structures. The complexity of the structure is justified when it brings an economic effect, i.e., when comprehensive structure is more economical than any single-purpose alternation.

In designing a new reservoir it is necessary to see to it

(a) that the future aim should not be difficult or impossible to attain;

(b) that the present utilization of the river should not be harmed; however, benefits can be eliminated if they are replaced by better ones (Vyšší Brod – Lipno, Dixence – Grand Dixence, Aswan – Sad el Aali, etc.).

The competition of the components of a water-management complex does not make it impossible to reach a comprehensive solution, it only makes it more difficult. According to the character of their need of water, the users can be divided into two groups:

(a) *users* who only use the water without consuming it or deteriorating its quality (hydro-power, navigation, fish farming, recreation, etc.);

(b) *consumers* who either completely or partly consume the water or deteriorate its quality (water supply, irrigation, etc.).

The water consumer greatly limits utilization of the source by others. The user limits the other components by changing the time pattern of the natural discharges. It is therefore more difficult to control the release which requires a larger reservoir volume.

If the water source cannot fully satisfy all those who need water or if its use is too expensive it is possible to

– bring water from an other source, e.g., the neighbouring catchment;

– exclude the least important or otherwise replaceable component of the complex, or limit the respective components according to their significance to attain the total optimum effect of water resource utilization.

In the multi-purpose reservoir both the economic effectiveness of the reservoir as a whole and the respective components of the water-management complex must be taken into consideration.

To solve this complicated economic problem, the effectiveness has to be proved when this comprehensive alternative is compared with an alternative solution which can be another multi-purpose scheme, a set of single-purpose reservoirs or any other form aimed at the same target. To determine the economic effect of the respective purposes, the investment and operation costs must be divided into individual components of the whole complex.

For the analysis of this economic problem many methodological approaches have been used. Often the analysis of the main component of the complex with a prevailing effect can be preferred symplifying thus the analysis. For the further development of the economic evaluation of the multipurpose reservoirs it is necessary:

- to use the more comprehensive methodology,
- to acquire the necessary technological and economic parameters,
- to seek the best possibilities, how to incorporate the intangible aspects in the analysis.