Chapter 10

Reservoir Sizing

The objectives of this chapter are:

- to explain the need of a reservoir and requirements of various uses of water,
- to discuss techniques for estimation for water yield at a site, range analysis, regulation regime function, etc.
- to explain methods for estimation of the size of the conservation storage, and
- to explain the methods for estimation of the size of the flood control storage.

Dams are constructed for two main functions. The first is to store water in the lake behind the dam to even out the fluctuations in river flow and match the availability with demand. If surplus water is available in a stream at a time earlier than its demand, it has to be carried over through storage from the period of surplus to the period of deficit. The purpose of regulation is to match the releases from the dam with the demand or to temporarily store high flows in the river to reduce flood damage. The second function of the dam is to create a hydraulic head of water (difference in height between the surface of a reservoir and the river downstream) in the reservoir upstream of the dam so that water can be diverted into a canal and flow due to gravity. The creation of storage and head allows dams to generate electricity; to supply water for agriculture, industries and households; to control flooding; and to assist river navigation by providing regular flows and drowning rapids. Other reasons for building large dams include fisheries and recreation activities, such as boating.

It is believed that the early Mesopotamians were perhaps the first dam builders. The Sumerians had built networks of irrigation canals in the plains along the lower Tigris and Euphrates rivers about 6500 years ago. Remains of dams which were built around 3000 BC as part of an elaborate water supply system in Jordan have been found. Earth and rock-fill dams had been built around the Mediterranean, in the Middle East, China, and Central America by the late first millennium BC. The remains of impressive dams and aqueducts in Spain are fine examples of the ingenuity of Roman engineers. South Asia has a long history of dam building. Long earthen embankments were built to store water for irrigation. The remains of the Indus Valley Civilization which flourished 4000 to 5000 years ago show that these people were familiar with many hydrologic principles and had built well planned networks of water supply and drainage works. The landscape of South India is dotted with a large number of very old small artificial ponds known as 'tanks' which are still in use as an important source of irrigation water. According to Morris and Fan (1998), the oldest reservoir in operation today is the Aftentang reservoir (storage 100 million m^3), constructed west of Shanghai during 589 to 581 BC. The oldest continuously operating dam still in use is the Kofini flood control diversion dam and channel constructed in 1260 BC on the Lakissa River upstream of the town of Tiryns, Greece, which it continues to protect.

The nineteenth century dams were mainly earthen embankments designed largely on the basis of thumb rules. Dam builders in the 19^{th} century had little streamflow or rainfall data, and few statistical tools to analyze whatever hydrologic data had been gathered. As a consequence, some of these dams failed. After the turn of the 19^{th} century, there were important developments in civil engineering. Consequently, the size of dams and power stations being built began to increase rapidly. The improvements in dam engineering allowed the high dams to be built and progress in turbine design increased the head at which the turbines could operate.

The International Commission on Large Dams, ICOLD (1988) has published details of the world's large dams (their website http://www.icold-cigb.org contains a lot of useful information). There were more than 36000 dams by 1986; many more have been constructed since then. The Asian continent accounts for more than 64% of all dams and China has built most of these. Engineers of the former USSR have built most of the large reservoirs; they are followed by engineers from Canada. More than 80% of the dams in the world are earth and rockfill type.

A dam contains a number of structural features other than the main wall itself. Spillways are used to discharge water when the reservoir level threatens to become dangerously high. Dams built across broad plains may include long lengths of ancillary dams and dykes. Weirs and barrages are constructed to divert river flow; they do not have significant storage and cannot effectively regulate flows. A weir is normally a low masonry or concrete wall. A barrage is a bigger (usually metallic) structure, often extending for hundreds of meters across a wide river.

10.1 NEED FOR RESERVOIRS

An important aspect of water resources development projects is planning and operation of reservoirs which are the most important component of a water resources project. Using a reservoir, the natural streamflow can be regulated so that the outflow follows the desired pattern. In the day-to-day life also, the concept of reservoirs is frequently used. For example, in the cities where the municipal water supply is erratic, residents use vessels to store water whenever there is running water in the taps and the stored water is withdrawn and used for various domestic needs when the taps go dry.

Due to the large size, the reservoir projects are capital intensive, i.e., they require huge amounts of money, manpower, land and other resources. Furthermore, these projects significantly affect the environment, population and economy of the region in which they are constructed. Since the financial resources available are usually limited, these should be carefully used to impart the maximum possible benefit to the national economy. Moreover, once the dams are in-place, it is not easy to undo or partially off-set their harmful impacts. Recently, some of the dams have been involved in various controversies which have resulted in frequent reviews, changes in design, and delay in construction resulting in huge cost over-runs. Due to these reasons, it is necessary that these projects are planned with utmost care and detailed examination of the issues involved.

The commonly used terms which are relevant to a reservoir are defined in Appendix A. A schematic diagram of a reservoir is given in Fig. 10.1.

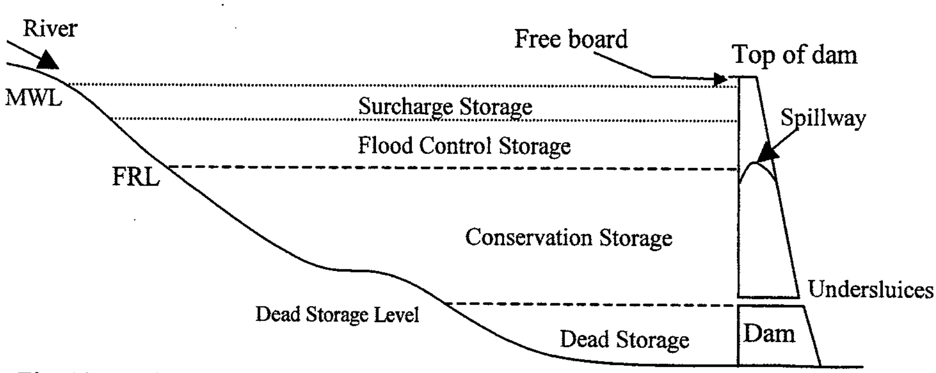


Fig. 10.1 Schematic diagram of a reservoir.

The time unit that is adopted for analysis of data for long-term planning is one year. To define year, one can follow the concept of calendar year, financial year, or water year. In water resources analysis, usually the concept of water year is used. The starting month of a water year varies from country to country, depending on the geographic location and climatic pattern. In general, a water year begins during the time of low flows. From a statistical point of view, the starting month should be chosen such that the coefficient of variation of annual streamflows is maximized while the autocorrelation is minimized (McMahon and Mein, 1986).

The principal function of a reservoir is regulation of natural streamflow by storing surplus water in the high flow season to control floods and releasing the stored water in the dry season to meet various demands. Generally, the major part of the annual streamflow is available during a few months of rainy season. But the demands for water arise all year round and therefore it is necessary to store the excess water in the rainy months so that it can be used when the natural streamflow is not sufficient to meet the demand. The water stored in a reservoir may be diverted by means of pipes or canals to far away places where it is needed; this diversion results in spatial changes of water availability. The water may also be kept in the reservoir and released later for beneficial uses resulting in temporal changes. In short, the aim of a reservoir is to match the temporal and spatial availability of water with demands. Depending on the magnitude of natural inflows and demands at a particular time, the reservoir storage is either built up or water is supplied from the storage.

As a result of storing water, a reservoir provides a water head which can be used for generation of electric power. The reservoir also provides an empty storage space for moderating inflow peaks. A reservoir also provides a pool for navigation to negotiate rapids, habitat for aquatic life and facilities for recreation and sports. It enhances scenic beauty, promotes afforestation, and supports wild life.

10.1.1 Classification of Reservoirs

Various classifications of reservoirs are possible depending on the purpose, size, and the storage space available in it. These are briefly discussed as follows.

Classification Based on Purposes

Depending on the number of purposes that a reservoir serves, a reservoir may be classified either as single purpose or multipurpose. A single purpose reservoir serves only one purpose. This purpose may be either a conservation purpose like water supply for domestic and industrial purposes, irrigation, navigation, generation of hydroelectric power, and recreation. The flood control purpose is a non-conservation in nature. A multipurpose reservoir is designed and operated to serve a combination of these purposes.

Classification Based on Size

Depending on the size, reservoirs are classified as major, medium or minor. These norms, however, vary from country to country. In India, if the gross capacity and the hydraulic head of the reservoir exceed $60*10^6$ m^3 and 30 m, respectively, the reservoir is classified as a major reservoir. If gross capacity lies between 10 and $60*10^6$ m^3 and the hydraulic head lies between 12 and 30 m, a reservoir is classified as a medium reservoir. Minor reservoirs have a gross capacity of less than 10 million m^3 and a hydraulic head of less than 12 m.

Classifications Based on Storage

Based on the storage space provided, a reservoir may be classified as a seasonal storage or over-year storage. A seasonal storage reservoir is designed to serve conservation purposes for periods of low flows. These reservoirs fill and spill frequently and are constructed on small tributaries to serve relatively small areas. An over-year storage reservoir is designed to serve for periods exceeding more than a water year. The storage in an over-year storage reservoir at the end of a water year is carried over to the next year. These reservoirs may neither be completely full nor dry every year.

10.2 CHARACTERISTICS AND REQUIREMENTS OF WATER USES

As a thumb rule, the bigger a reservoir is, the more are the purposes that it can serve. The major purposes for which a reservoir is used and the functional requirements for these are discussed below. The irrigation requirements have been described in greater detail in Chapter 9. The hydropower generation is discussed in greater detail in a later section.

a) Irrigation

Irrigation demands are consumptive and only a small fraction of the water supplied is available to the system as return flow. These requirements have direct correlation with rainfall in the command area. Irrigation requirements are seasonal in nature and the variation largely depends on cropping patterns in the command area. In general, demands will be small during the wet season and large during winter and summer months. The average annual demands remain more or less steady unless there is increase in the command area or large variation in the cropping pattern. The safety against droughts depends on the available water in the reservoir and hence it is desirable to keep as much water in storage as possible consistent with current demands.

b) Municipal and Industrial Water Supply

Generally, water requirements for municipal and industrial purposes show less change through the year, more so when compared with irrigation and hydroelectric power. The water requirements increase with time due to growth and expansion. The seasonal peak of the demand is observed in summer. For the purpose of design, a target value is arrived at by projecting population and industrial growth. The supply system for such purposes is designed for a high level of reliability.

c) Hydroelectric Power

Water is a renewable source of energy. The hydroelectric power generation is a non-consumptive use of water because after passage through the power plant where its mechanical energy is converted to electric energy, the same water can again be utilized for other uses downstream. Due to this feature, hydroelectric projects are frequently multi-purpose. As a result of research and development in turbine technology, efficient turbines with capacities varying from several hundreds of MW to a few MW have been developed. Therefore, one now comes across mega hydropower projects, providing power to a big region, to micro projects, catering to the needs of a small village. It is estimated that one-quarter of the electrical energy generated in the world is from hydropower. Some advantages of hydropower generation are:

- This is a renewable source of energy, the sun being the prime mover of water cycle. As no payment is made for the input, the production is free from inflation.
- The hydropower plants do not require much outlay on account of operation and maintenance, and have a long life.
- The hydropower generation does not pollute the environment; no heat is produced and no harmful gases are released.
- The hydropower power plants work at a very high efficiency (say up to 90%), whereas the thermal power plants work at a comparatively low efficiency.

- The plant can be started or shutdown in a short time, with no wastage of water.

The electric power demands usually vary seasonally and to a lesser extent daily and even hourly. The degree of fluctuation depends on the type of loads being served, viz. industrial, municipal, and agricultural. For example, in case of municipal areas, the hydroelectric demands are at maximum during the peak summer months. Furthermore, during the course of a day, two demand peaks are observed, one in the morning and another in the evening. The hydroelectric power plants are usually part of a regional or national grid and their operation is governed by their role in the grid. The hydropower generation aspect is discussed in greater detail in Section 10.5.

d) Flood Control

Flood control reservoirs are designed and operated to moderate flood flows that enter into them. The flood moderation is achieved by storing a part of inflows in the reservoir and releasing the balance. The degree of flood attenuation or moderation depends on the empty storage space available in the reservoir when the flood impinges on it. The achievement of this purpose requires the availability of empty storage space in the reservoir. As far as possible, the releases from the storage are kept smaller than the safe capacity of the downstream channel.

e) Navigation

Storage reservoirs may also be operated to maintain a stretch of downstream river navigable. The requisite depth of flow in the navigation section is maintained by releasing water from the dam. The demand for water for this purpose depends on the type and volume of traffic in the navigable waterways. The water requirements for navigation show a marked seasonal variation. There is seldom any demand during the wet period when sufficient depth of flow is available in the channel. The demands are at a maximum in the dry season when large releases are required to maintain the required depth.

f) Thermal Power generation

Water is also an important input in thermal power generation where it is used for cooling purposes. The simplest arrangement, known as once-through cooling, consists of diverting the water from the source to the power plant where it is used to cool the condensers and the heated water is returned to the source. Although this method has many advantages, the main being the least cost of construction, it is being gradually discarded due to thermal pollution of the receiving waters. Most new power plants use evaporative cooling tower systems. In such systems, a tall cooling tower is constructed and the water, after cooling the condensers, is passed through an air stream, cooled, and recycled to the condensers.

g) Recreation

The benefits from recreation are derived when the reservoir is used for swimming, boating, skiing and other water sports and picnic. Usually the recreation benefits are incidental to other uses of the reservoir and rarely a reservoir is constructed solely for recreation. The recreation activities are best supported by a reservoir which remains nearly full during the recreation season. Large and rapid fluctuations in the water level of a reservoir are harmful from a recreational point of view because they can create marshy land near the rim of the reservoir.

h) Minimum flow maintenance

Many times, it is necessary to release a certain minimum amount of water in the river below the reservoir from water quality (dilution of pollution) or environmental considerations. The release under this head vary seasonally and may get the highest priority.

10.3 RESERVOIR PLANNING

Since reservoirs are capital-intensive projects, it is necessary that these projects are carefully planned and executed. All the available data should be analysed and if necessary, further information should be gathered so that the best decision is taken with respect to location, size and type of structure and auxiliary facilities. First, the best site for the dam and reservoir is selected and then a number of investigations are carried. These are described in what follows.

10.3.1 Site Selection Criteria for a Reservoir

The following factors should be kept in mind while selecting the site for a reservoir:

1. The reservoir site should be such that the leakage of water through the ground is minimum. Sites having permeable rocks reduce the water tightness of the reservoir. The rocks which allow less passage of water include shales, slates, schists, gneiss, and crystalline igneous rocks such as granite.
2. A suitable site for the dam must exist. The dam should be founded on sound watertight rock base, and percolation below the dam should be minimum. The cost of the dam depends on the suitability of a site and is often a controlling factor in the site selection.
3. The reservoir basin should have a narrow opening in the valley so that the length of the dam is the least possible.
4. The cost of the real estate for the reservoir, including road, railway, rehabilitation and resettlement etc. must be as small as possible.
5. The topography of the reservoir site should be such that it has adequate storage capacity without submerging excessive land and other properties.
6. The site should be such that a deep reservoir is formed. A deep reservoir is preferable to a shallow one because of the lower cost of the land submerged per unit of capacity, less evaporation losses due to reduction in the water spread area, and less likelihood of weed growth.
7. The reservoir site should be such that it avoids or excludes water from those tributaries which have a high concentration of sediments in water.
8. The submergence area should not contain, to the extent possible, sites of

archeological importance, major towns, forested area, and habitats of rare species.

9. The soil and rock mass at the reservoir site must not contain harmful minerals and salts.

It is often not possible to find a site which satisfies all of the above conditions. In such cases, the planner uses discretion to choose the site which best meets the project objectives.

10.3.2 Investigations for Planning a Reservoir

The various investigations required for reservoir planning are described below:

Engineering Surveys

The area around the potential dam site is surveyed in detail and a map with a small contour interval (of the order of a few meters) is prepared. From the map, the following are prepared:

a) area-elevation curve,
b) storage-elevation curve,
c) details of the land and property likely to be submerged, and
d) suitable site to locate the dam.

Conventionally, the topographic maps are prepared by carrying out a plain-table survey of the area. However, these days the maps can be quickly prepared using the remote sensing techniques. This technique saves time and cost is very handy for areas which are difficult to access. This technique has been described in Chapter 3.

Area-elevation and Storage-elevation Curves

Once the site of the dam is finalized, a map on the reservoir area with a small contour interval, say 1m or so, is prepared. Starting with the lowest contour, the areas enclosed by the successive contours can be determined with a planimeter. Clearly, as elevation of a contour increases, the enclosed area also increases. Thus, a curve may be drawn with elevation on the X-axis and area on the Y-axis. Such a curve for a reservoir is shown in Fig. 10.2. The contour plan also shows the water spread corresponding to the maximum water level in the reservoir. This information is used to determine the area likely to come under submergence.

The reservoir capacity or the volume of storage corresponding to a given water level may be calculated by the trapezoidal formula. Thus, if A1 and A2 are the areas between two successive contours, and h is the contour interval, the intermediate storage volume V can be calculated using the formula:

$$V = (A1 + A2)*h/2 \tag{10.1}$$

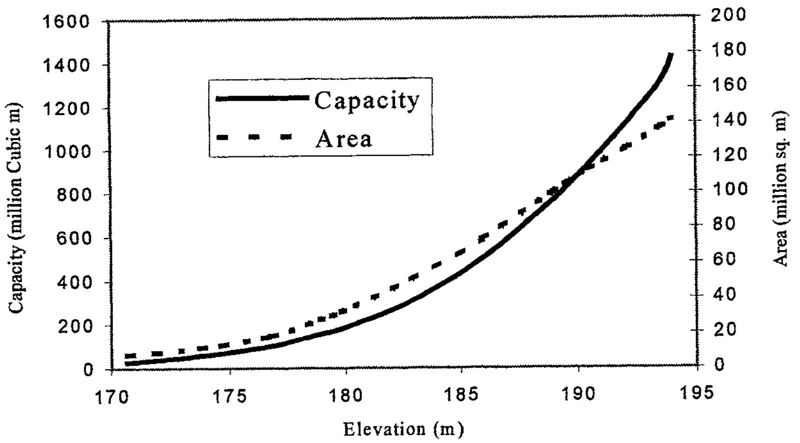


Fig. 10.2 Elevation-area-capacity curves of Dharoi reservoir.

The total reservoir capacity at a given elevation is computed by adding the incremental volumes up to that elevation. The storage volumes corresponding to various water-surface elevations may be calculated and a curve, called capacity curve, may be plotted between elevation and storage as shown in Fig. 10.2.

Geological Investigations

In all major civil engineering projects, geological surveys are critically important. The geological investigations are required to give detailed information about the following:

1. hydro-geology of the area;
2. suitability of foundations for the dam;
3. geological and structural features, such as folds, faults, fissures etc. in the reservoir basin;
4. water tightness of reservoir basin;
5. location and extent of permeable and soluble rocks, if any;
6. groundwater conditions in the region; and
7. location of quarries for materials required for dam construction and quantities available from them.

The geological investigations cost little in comparison to the total cost of the project; typically it may amount to between 0.25 to 1 per cent of the project cost. This relatively small amount represents a valuable insurance against difficulties which might arise during construction. There have been instances where discovery of adverse geological features during construction led to disruption of work and time plus cost overruns.

An important requirement for the reservoir site is that there should be no danger of large leakage when the ground is under pressure from the full head of water in the reservoir. The geology of the dam site is important from the point of view of suitable foundation for the dam. The nature of sub-surface geology should be explored by trial bores or various means of geophysical exploration. The geology of the catchment area should also be studied since it affects runoff and percolation.

Hydrological Investigations

These investigations are important in reservoir planning as the reservoir size, height of dam, capacity of the irrigation canals, the installed capacity of the power house, etc. depend on the available water. The hydrological investigations can be divided in the following heads:

a. Study of runoff pattern at the proposed dam site to determine the water availability and storage capacity required to meet the target demands,
b. determination of the hydrograph of the worst flood for spillway design,
c. estimation of evaporation and other losses from the reservoir particularly in an arid area,
d. sedimentation studies to determine the sediment inflow into the reservoir and its impact on the reservoir performance, and
e. simulation studies to study the performance of the reservoir under a given inflow series and demand pattern.

For reservoir planning, the first step is the correct assessment of water availability at the site. This requires a sufficiently long sequence of data; the data length depends on the type of storage, type of project, and variability of flows. An analysis carried using a longer period of data will give more reliable results. A longer data series would be required for over-the-year-storages. A comparatively shorter length will suffice for within-the-year storage where the spill occurs almost every year and the critical period is of the duration of a few months. Different agencies have issued guidelines regarding the minimum length of data required and one such guideline is as under:

Table 10.1 Minimum length of data required for various projects.

Type of Project	Minimum data length for use in analysis
Diversion projects	10 years
Within the year storage projects	25 years
Over the year storage projects	40 years
Systems having a combination of the above	Depending on the predominant element

The flow sequences required for planning of projects need to be prepared for an appropriate time unit so that the simulation studies can have a desired resolution. The number and type of additional hydro-meteorological stations to be set up in the catchment and the command areas, if any, are decided during hydrologic investigations. While fixing the location of additional stations, future requirements for the operational stage of the project should also be considered.

After assessing the data required and availability, various techniques are used to extend/generate long-term flow sequences (if necessary) for proper evaluation of water availability and project planning. Since rainfall data are normally available for a longer period than runoff data, it is common to extend the runoff data using rainfall data. The water availability analysis is described in a later section.

Reconnaissance (Preliminary) Investigations

The main purpose of such investigations is to screen out the inferior alternatives and to decide on further data which need to be collected for detailed feasibility investigations of the remaining selectable alternatives. A reconnaissance survey will identify the scope of a project plan with respect to its geographical location, project functions, approximate size of its various components, likely problem areas and time and cost of conducting feasibility investigations.

Actually, a reconnaissance investigation is a preliminary version of a feasibility investigation carried out in a short time and with less accuracy. It considers all the physical, engineering, economic, environmental and social aspects related to the project. It is usually conducted with the available data. Collection of some new data, if considered necessary for reconnaissance, is made by surveys. These may include the simple cross-section (instead of detailed topography) of the stream at dam site, surface investigations of geological conditions at the dam site, sub-surface explorations for dam foundation, quality and quantity of available construction materials, and so forth. Preliminary designs are made by using short-cut methods (e.g., empirical curves, tables, and previous experience). Cost and benefits of the project are also estimated. Based on the results of reconnaissance, a set of project plans is selected for subsequent detailed investigation.

Feasibility Investigation

The aim of feasibility investigations is to ascertain the soundness and justification, or otherwise, of different alternative plans chosen after carrying out preliminary investigations. The analyses need to be of high accuracy and dependability so that the reliability of results, on the basis of which the final selection of the project plan is made, is not questioned. However, completion of feasibility investigation does not mean the end of the project planning. Minor changes may be required for various reasons before construction and even during construction.

The first step in the feasibility investigation is to collect or update the basic data of different types. The accuracy and reliability levels of these data must be consistent with the degree of accuracy required for feasibility justifications. The basic data for dams and reservoirs include topographic surveys of sites, information on streamflow and design flood, land costs, reservoir clearing costs, communication facilities, climatic conditions affecting construction, fishery and wildlife to be preserved, construction material, foundation conditions of dam site and reservoir area, availability of trained manpower and environmental and other considerations. The facilities and appurtenances necessary for the functioning of the project must be specified and considered while making cost estimates.

Pre-construction Investigations

These investigations are carried out if the time elapsed between the feasibility investigation and commencement of construction is large. It is essential that any new information which might have become available during the intervening time is incorporated in final designs. For example, a flood of large magnitude might have occurred during this intervening period and this may necessitate changes in the design flood for the project and consequently the design of spillway and related structures. Best (1998) has provided a discussion on investigations for dam construction.

10.4 ESTIMATION OF WATER YIELD USING FLOW DURATION CURVES

A popular method to study the streamflow variability is the flow duration curves. A flow-duration curve of a stream is a plot of discharge against the percent of time the flow was equaled or exceeded. This curve is also known as discharge-frequency curve. To prepare it, streamflow data are arranged in the descending order of discharges using class intervals. The data used can pertain to any time step, daily, weekly, ten-daily or monthly. If N number of data points are used, the plotting position of any discharge (or class value) Q is

$$P = m/(N+1) * 100\% \tag{10.2}$$

where m is the order number of the discharge (or class value), and P is the percentage probability of the flow magnitude being equaled or exceeded. The plot of the discharge Q against P is the flow duration curve (Fig. 10.3). An arithmetic scale, a semi-log scale, or log-log graph may be used, depending on the range of data and use of the plot. The flow duration curve represents the cumulative frequency distribution and shows the streamflow variation of an average year. The ordinate Q_P at any percentage probability P represents the flow magnitude in an average year that can be expected to be equaled or exceeded P % of the time and is termed as P % dependable flow. In a perennial river Q_{100} (100% dependable flow) is a finite value, e.g., it is about 60 units in Fig. 10.3. In an intermittent or ephemeral river, streamflow is nil for a finite part of a year and as such Q_{100} is zero.

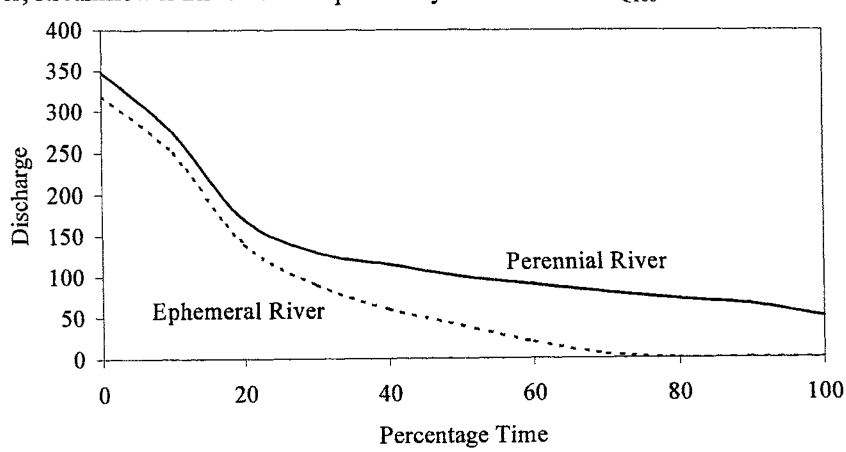


Fig. 10.3 Typical flow duration curves.

Some important characteristics of a flow duration curve are:

1. The slope of a flow duration curve depends on the time interval of the data selected. For example, a daily streamflow data gives a steeper curve than a curve based on the monthly data for the same stream. This is due to the smoothening of small peaks in monthly data.
2. The presence of a reservoir on the stream considerably modifies the virgin-flow duration curve depending on the nature of flow regulation. Fig. 10.4 shows the typical reservoir regulation effect.
3. The virgin-flow duration curve plots as a straight line on a log probability paper at least over the central region. From this property various coefficients expressing the variability of flow in a stream can be developed to describe and compare different streams. A steep slope of the flow-duration curve indicates a stream with a highly variable discharge. On the other hand, a flat slope indicates a slow response of the catchment and a small variability. At the lower end of the curve, a large flat portion indicates considerable base flow. A flat curve on the upper portion is typical of river basins having large flood plains and also of rivers having large snowfall.
4. The chronological sequence of occurrence of the flow is masked in the flow-duration curve. A discharge of say, 1000 cumec, in a stream will have the same percentage P whether it occurred in January or June. This aspect, a serious handicap, must be kept in mind while interpreting a flow-duration curve.

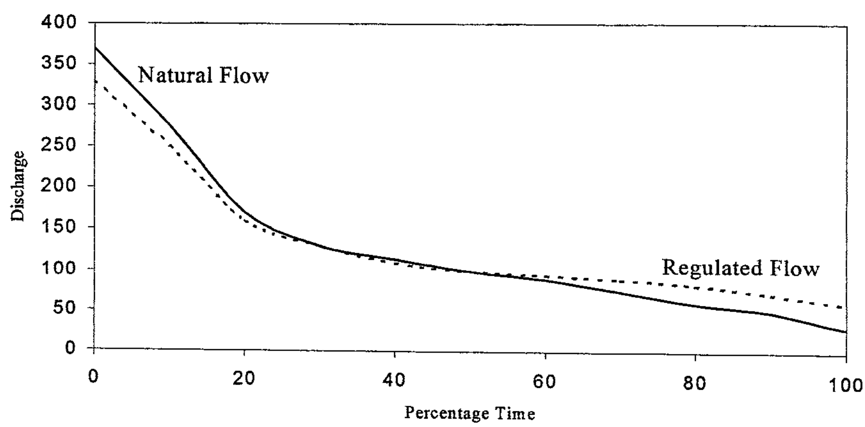


Fig. 10.4 Reservoir regulation effect on flow duration curve.

10.4.1 Procedure to Prepare a Flow Duration Curve

The Institute of Hydrology (1980) has outlined the procedure for estimating the flow duration curve depending on the availability of data at or near the site of interest. The guidelines suggested for a given length of record are given below:

More than ten years of records: Such records need no adjustment or standardization as this period of data will probably provide a sufficiently accurate flow duration curve.

Two to ten years of records: For this length of records, divide the daily flow data by the average flow over the period of record before analysis. This overcomes to a great extent the departures due to wet or dry years. The conversion to the long-term flow duration curve is made using an estimate of long-term average flow.

Less than two years: This length of record may be treated as short and some indirect approaches are used for flow duration curve computations. These approaches are based on the use of catchment characteristics.

Preparing Flow Duration Curves from Daily Flow Data

The flow duration curves from daily flow data may be prepared using the following steps:

i. Choose a class intervals (CI) such that about 25 to 30 classes are formed.
ii. Assign each day's discharge to its appropriate CI.
iii. Count the total number of days in each CI.
iv. Cumulate the number of days in each CI and get the number of days above the lower limit of each CI.
v. Compute the probabilities of exceedence by dividing the quantities obtained from step (iv) by the total number of days in the record (for example 365 if one year record is being used to construct flow duration curve).
vi. Multiply the probabilities of exceedance obtained from step (v) by 100 to get the percentage exceedance.
vii. Plot the probabilities of exceedance in percent against the corresponding lower bound of CI on linear graph paper. Sometimes the flow duration curve better approximates as a straight line if the log normal probability paper is used in place of linear graph paper.

Sometimes a flow duration curve is prepared for duration other than the base duration for which data are available. For example, the daily data at a site might be available but the flow duration curve of 10-day flows might be needed. The flow duration curves for durations other than the base duration can be prepared as follows:

i. Derive a hydrograph whose values are not simply daily discharges but are average discharges over the previous D days (D-duration of flow duration curve). It is equivalent to the outcome of passing a moving average of D-day duration through the daily data. Generally 1, 5, 7, 10, 30, 60, 90, 180 and 365 days are adopted as the standard values for D.
ii. Plot the flow duration curve from the data of discharge hydrograph derived in step (i) using the procedure described previously. Fig. 10.5 illustrates typical flow duration curves for different durations.

10.4.2 Use of Flow Duration Curves

The flow duration curves are used to estimate the dependable flows of a river for various reliabilities. In various countries, guidelines have been formulated for planning of river valley projects for different purposes. For example, according to the practice in India, irrigation projects are planned using 75% dependable flow. Hydropower and drinking water

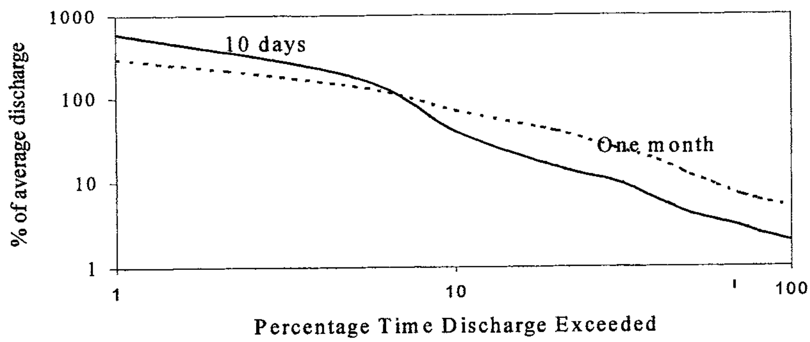


Fig. 10.5 Flow duration curves for different durations.

projects are planned with 90% and 100% dependable flows, respectively. The 90% dependable flow is also used as a measure of ground water contribution to stream flow. This same value can also be used as a measure of run-of-the-river hydropower potential. Other important uses of flow duration curves are:

1. to evaluate the characteristics of the hydropower potential of a river,
2. to design drainage systems,
3. for flood-control studies,
4. to compute the sediment load and dissolved solids load of a stream, and
5. to compare adjacent catchments with a view to extend streamflow data.

10.5 HYDROPOWER GENERATION

Since time immemorial mankind has been using energy of water falling from a height to perform useful works, for example, water wheels to grind grains. Hydroelectric energy is produced by converting the mechanical energy of falling water into electrical energy by a turbine and generator. Hydropower is a renewable and a clean power source which does not produce any air or water pollution and is one of the most efficient ways to generate electricity. In multi-purpose projects, generation of hydro-electric energy is combined with other uses, such as irrigation, water supply, etc. resulting in tremendous savings of money and other resources. Another advantage of hydropower over other forms of electricity generation is that reservoirs can store water during times of low demand and can quickly start generating during the peak hours of electricity use. Thermal power plants take much longer to start up from cold than hydropower plants.

With respect to types of site development, there are four major classifications of hydroelectric projects: storage, barrages, run-of-river, and pumped storage. Storage projects usually have heads in the medium to high range (greater than 25 m) and have provisions to store relatively large volumes of water during periods of high streamflow to provide water for power generation during periods of deficient streamflow. The power house is commonly located at the toe of the dam, although in some cases it might be away from the dam.

Peaking operation is frequently associated with storage projects and this requires large and sometimes rapid fluctuations in releases of water through the generating units. It is often necessary to provide facilities to even out the fluctuations in the discharge if rapid changes of discharge below the project are not desired. For example, such an arrangement exists in the Bhakra Nangal project in India where a small barrage (Nangal barrage) has been constructed at some distance downstream of the Bhakra dam (a major power project).

A barrage, also known as pondage, has a very small storage capacity. It can regulate the flow only up to minor extent and generate power according to the weekly or daily variation of the load. Hence, the tail water fluctuations are usually quite large, particularly in peaking operations. If the cycle of peaking operation is a single day, the pondage requirements are based on the flow volume needed to sustain generation for 12 hours. If more storage capacity is available and large fluctuations in the reservoir level are permissible, a weekly cycle of peaking operation may be considered. Since industrial and commercial consumption of power is significantly lower on week ends than on week days, an "off-peak" period is created from Friday evening until Monday morning. During this period, water can be accumulated in the pondage for later use.

Run-of-river plants have little or no storage and, therefore, must generate power from streamflow as it occurs with little or no benefit from at-site regulation. These projects generally have productive heads in the low to medium range (5 to 30 m) and are quite frequently associated with navigation or other multipurpose developments. For a base-load run-of-river project to be feasible, the stream must have a relatively high baseflow. Sometimes, the falls in irrigation canals are also used to generate energy. Because of (near) absence of storage, there is usually very little operational flexibility in these projects. The existence of upstream storage project(s) may make a run-of-river project in the lower part of the basin feasible where it would not otherwise be. But the storage projects must provide a regulated outflow that is usable.

Pumped Storage Schemes

Pumped storage projects consist of a high level forebay where inflow or pumped water is stored until it is needed for power generation and a low level afterbay where the power releases are stored. These projects depend on pumped water as a partial or total source for generating electric energy. The pumping and generation are done by units composed of reversible pump turbines and generator motors connecting the forebay and afterbay. The water is pumped from the afterbay to the forebay when the normal power demand is low and released from the forebay to the afterbay to generate power when the demand is high. Such projects derive their usefulness from the fact that the demand for power is generally low at night and on weekends and therefore, pumping energy at a very low cost will be available from idle generating facilities. The feasibility of pumped storage developments arises from the need for relatively large amounts of peaking capacity, the availability of pumping energy at a cheap rate and a load with an off-peak period long enough to permit the required amount of pumping.

There are three types of pumped storage development: diversion, off-channel, and in-channel. The diversion type of development usually consists of pumping in one basin to a forebay on or near the divide between that basin and an adjacent basin and it does not recirculate the water between the forebay and afterbay. The water is released through generating units into an afterbay located in the adjacent basin. The advantages are that it may be possible to pump against a head that is very small in relation to the head for generating, provided a source of water for pumping can be located at an elevation that is not too far below the forebay. This scheme has the disadvantage that separate pump-motor and turbine-generator units are required, whereas a single reversible unit is used in the other two types of pumped storage development.

The off-channel type of pumped storage development is most suitable when a forebay site exists on a hill above a stream where an afterbay can be constructed. The head differential should be large and the forebay site should be close to the afterbay to avoid head loss and reduce construction costs. The water requirement to support this type of development is not large after the initial supply has been provided. Since the system primarily recirculates water, it is necessary to provide water only to replace losses due to evaporation and leakage.

In the in-channel type of pumped storage development, the reservoir of a conventional power project is used as a forebay. The afterbay could be a reservoir from a downstream project or a reservoir provided solely to serve as afterbay. This type of development is more attractive if the cost of the afterbay is shared with other purposes. In an in-channel pumped storage project, the maximum possible amount of water is pumped back from the afterbay to the forebay during low flow period. During the less severe dry periods, only a part of the water that is used to generate power is pumped back into the forebay. During periods of high streamflow, none of the water is pumped back. This type of project is most valuable when there is a large difference between the streamflow quantities in low-flow periods and the average conditions because the reversible mechanism can add substantial firm energy with attendant pumping costs that are far less than what would be incurred with other types of development.

10.5.1 Components of Hydropower Projects

In a storage project, the reservoir behind the dam stores water that is used to generate electric power. The portion of the reservoir that is immediately upstream of the intake structure is known as forebay. The water is withdrawn from forebay through an intake structure and is carried to the power house through penstocks. A penstock is a conduit to carry water from the forebay to turbines and gates and valves are installed to control the water flow. Depending on the site conditions, open channels or tunnels may instead be used. A surge tank is constructed to handle the problems of water hammer. The turbines and generators are installed in the power house. The water comes out of turbines through draft tube and joins the tail water.

The term 'static head' (see Fig.10.6) denotes the difference between the water surface elevation in the forebay and the elevation of tail water (TWL). The 'net head' is static head less losses in the penstock:

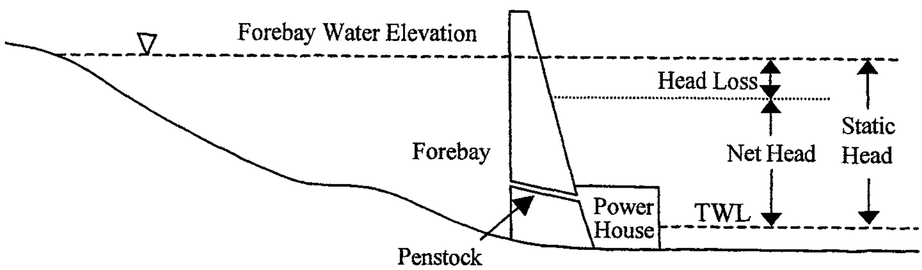


Fig. 10.6 Terms related to head in a hydropower plant.

Net head = Static head – Losses
= Water surface elevation in forebay – TWL - Penstock losses (10.3)

Although the head is usually related to the dam height, a low dam can yield a high head if the turbines and generators of powerhouse are located some distance downstream of the dam. The amount of power generated is a function of discharge and the hydraulic head. It can be computed as

$$P = 9.817QH\eta \qquad (10.4)$$

where P is the electric power in kW, Q is the discharge through power plant in m^3/s, H is the net head in m, and η is the overall efficiency of the power plant expressed as a ratio (usually about 0.85). The overall efficiency of the power plant is obtained by multiplying the turbine efficiency with the generator efficiency. The hydroelectric power generation depends on the volume of water passing through turbines and the effective head. Thus, the same amount of power can be produced by releasing more water at a low head or less water at a high head. Furthermore, it is better to construct these plants in hilly areas where steep slopes provide high heads.

Load of Hydropower Projects

With respect to the type of load served, hydropower projects can be classified in two categories: base-load plants and peaking plants. Base-load projects generate power to meet the base-load demand (the demand that exists 100 % of the time). Usually, the base-load is served by thermal generating facilities. But if there is a relatively abundant supply of water with a high degree of reliability and fuel is relatively scarce, hydroelectric projects may also be constructed to serve the base-load.

Peaking plants supplement base-load generation during periods of peak power demands. These plants must have sufficient capacity to satisfy peak demands of a system and water should be sufficient to provide the peaking support for as long and as often as is needed. In general, a peaking hydroelectric plant is desirable in a system that has thermal generation facilities to meet base-load demands. The hydroelectric generating facilities are particularly adaptable to the peaking operation because their output can be changed rapidly. Further, the seasonal variations in streamflow can be easily offset by providing storage.

The demand for electrical energy is known as load. The ratio of the average power demand to peak power demand for the time period under consideration is known as a load factor and this is computed on a daily, weekly, monthly or annual basis. Thus,

$$\text{Load factor} = [\text{Average power demand}] / [\text{Peak power demand}] \tag{10.5}$$

an appropriate time unit is chosen in this equation.

The generation of energy is limited by the installed capacity of the power plant. The term firm energy is used to denote the energy that is available with 100% reliability. The concept is similar to firm power. The energy that is available over and above the firm energy is known as secondary energy. The generation of energy depends on the installed capacity of the power plant. Depending on the configuration of the system of which the hydropower plant is a part, it may be operated to meet the base load or the peak load. It is more efficient to operate the hydropower plant to meet the peak load, because these plants can be put on and off at a short notice and there is no wastage of resources. However, there might be other considerations in operation.

10.5.2 Estimating Hydropower Potential and Demand

Analogous to the concept of firm water, firm power is the maximum quantity of power that can be guaranteed to be delivered 100% of the time according to some prescribed distribution. The hydroelectric power potential is determined on the basis of the critical period as indicated by the historical streamflow record. The critical period is a function of the power demand, streamflow, and available storage. If a project serves more than one purpose and if, in serving another purpose, some of the storage or streamflow is not available for power production, the streamflow data should be adjusted to reflect the "loss". Losses, such as evaporation, leakage, and station use, must also be deducted from the available flow before calculating the potential energy. The amount of power generated over a time, or energy, is expressed in kilowatt-hour (kW-hr). It can be computed as:

$$KWHR = 9.817QHT\eta \tag{10.6}$$

in which KWHR is the hydropower generated during the period in kw-hr and T is the number of hours in the period.

While computing the average head, the tailwater elevation should represent average conditions during the time when power generation actually occurs. For example, in a peaking project that usually generates power at or near the installed capacity for a short duration, the tailwater elevation should correspond to the discharge at installed capacity rather than to the average discharge. Likewise, if there are releases that do not pass through the generating units but which significantly affect the tailwater, the tailwater elevation should reflect the combination of power releases and other releases.

Two methods are used to estimate the hydropower potential at a given site: The Flow Duration Curve method and Sequential Streamflow Routing (SSR) method. In the first method, the flow duration curve at the site is the basic input. The net head for various discharges is estimated. Using the data of the usable range of flow duration curve and head vs. discharge data, a head-duration curve is developed. The hydropower equation is used to estimate the power generated at many points on the flow duration curve and a power duration curve is developed. The average annual energy (AE in kW-hr) and the dependable capacity (DC in kW) can now be calculated:

$$AE = 87.6 \int_0^{100} P dp \tag{10.7}$$

$$DC = 0.01 \int_0^{100} P dp \tag{10.8}$$

where P is power in kW and p is the percent of time. The major advantage of this method is that it is simple and fast. However, it cannot take into account the installed capacity and key project features, such as intake and power plant characteristics, etc.

In the SSR method, the time step size and period of analysis is chosen and the operation of the reservoir is simulated. For each time period, the reservoir outflow is computed using the continuity equation and other constraints on operation. The amount of energy generated corresponding to this outflow and head is calculated using eq. (10.6). The process is repeated for all the time steps. Now the average annual, monthly etc. generation as well as firm energy can be computed. An advantage of this method is that it can take into account the reservoir characteristics as well as power plant features. The results of this method are more realistic as compared to the flow duration curve method.

Knowing the average head, the volume of water needed to generate the desired amount of energy for a given time period can be computed using eq. (10.6). The summation of volumes for various periods will give the total volume required.

Example 10.1: Compute the volume of water required to generate 4.75 MW-hr of electric energy for one day if the average head is 95.0 m and efficiency of the power plant is 0.85.

Solution: The energy to be generated is 4750 kW-hr and T =24. From eq. (10.6)

$$9.817*Q*95.0*24*0.85 = 4750$$

or $Q = 0.2497$ cumec.

Volume of water required for one day = 0.2497*24*3600 = 21574.08 m^3.

10.6 RESERVOIR LOSSES

A portion of the water stored in a reservoir is lost and is not available for beneficial use due to various processes. The major causes of the loss of water are evaporation, leakage through the body of the dam and groundwater flow.

10.6.1 Evaporation Losses

The loss of water due to evaporation depends on the nature of the evaporating surface and meteorological factors (See section 2.5.1 for detail). The factors affecting the evaporation process are radiation, wind speed, and vapor pressure of the air overlying the surface. The amount of evaporation also varies with latitude, season, time of day, humidity, and condition of sky. It is difficult to categorically express the relative effect of the controlling meteorological factors. If radiation exchange and all other meteorological elements were constant over a shallow lake for a considerable time, the temperature of water and evaporation would become constant. If the wind speed were suddenly doubled, the rate of evaporation would also be double for some time.

The quality of water in a reservoir also affects evaporation to a small extent. This reduction takes place because dissolved solids reduce the vapor pressure for evaporation, the temperature of water rises and this partially offsets the effect of reduction in vapor pressure. Moreover, any foreign material which affects the reflective property of the water surface affects evaporation.

A pan evaporimeter is commonly used to estimate evaporation from a lake. The pans can be installed in three ways: on the land surface, sunken in ground, and floating on water surface. The pans installed on or above the ground surface show a little higher evaporation since extra heat is absorbed by the side walls. The main advantages of surface pan are economy and ease of installation and maintenance. An estimate of the depth of evaporation can be obtained by multiplying the pan evaporation by the pan coefficient.

The evaporation from a reservoir can be best approximated by a pan floating on the lake surface. However, the installation and maintenance expenses are quite large. Observation of evaporation data is difficult and many times, splashing takes place which renders the records unreliable. Due to these reasons, these pans are not very common in use.

Evaporation losses from a reservoir are depend on the water spread area and the rate of evaporation. These are expressed in terms of water depth. The depth of water lost by evaporation may typically be 100 to 200 cm annually. Evaporation in hot months is two to five times that in winter months. In humid areas, low values, say about 50 cm, are observed while it can go up to about 350 cm in arid climates. Due to significant loss of water in arid climates, extensive efforts have been made to control the same. In India, efforts have been made to control evaporation losses by laying layers of mono-molecular chemical films on the surface of the reservoir. These films try to retard evaporation by curtailing exchange of energy between water and atmosphere. However, these films have not been very successful because they are expensive to lay and easily break due to wind and waves.

Example 10.2: The water spread area of a reservoir is 15 sq. km and the evaporation from a pan (pan coefficient =0.7) at the dam for a day is 1.0 cm. Compute evaporation loss from the reservoir.

Solution: Evaporation loss = 15*1000*1000*0.01*0.7 = 105000 m^3.

10.6.2 Seepage Losses

A reservoir also exchanges flow with aquifers though the magnitude is small compared to the surface water inflow. The amount of this flow depends on the physiographical features and soil characteristics in the vicinity, and the position of water table. Assuming homogeneous conditions, the flow can be computed by the Darcy law.

Seepage losses are difficult to estimate and are not important in most cases. Seepage is generally more when the reservoir is underlain by porous strata having ample outlets beneath the surrounding hills or under the dam. Absorption losses may be significant in the initial stages but gradually reduce as the soil pores become saturated. Generally the reservoir banks are not so permeable as to cause significant leakage. Where the banks have continuous seams of porous strata or are made of fractured rock formations, pressure grouting is done to seal the fractured rock. The water lost from the reservoir to the ground water through seepage and passages in the reservoir bed is not amenable to measurement. However, an assessment of the losses can be made by considering the inflow, the outflow, and precipitation over the reservoir and evaporation loss and then labeling the unaccounted loss of water due to seepage and absorption. Evidently, the accuracy of the determination depends on the precision with which other variables of the equation are estimated.

10.6.3 Leakage through Dam

This component of outflow consists of the loss of water from the reservoir on account of leakage through the body of the dam as well as through gates and spillways. It is not easily possible to relate these losses with a measurable quantity. For example, the losses through the gates or valves of undersluices depend on their design, installation and maintenance. A simplifying assumption is that these losses linearly vary with the reservoir level. In general, the amount of water lost due to these reasons varies between 0.5% to 4% of the total discharge through the structure.

10.6.4 Water Balance of a Reservoir

The water balance equation for a reservoir is nothing but the mass balance or continuity equation. This equation states that the sum of inflow and outflow components and change in storage (with appropriate signs) must be zero over a given time interval. This equation can be expressed as:

$$I_S + I_G + P - E - Q - L - \Delta S \pm \delta = 0 \tag{10.9}$$

where I_S is the surface water inflow into the reservoir; I_G is the ground water inflow into the reservoir; P is the precipitation on the surface of a reservoir; R is the release from the reservoir; E is the evaporation from the reservoir; L is the storage loss including seepage, etc.; ΔS is the change in reservoir storage during the period of computation; and δ is the error term. All the terms are expressed either in volume units or depth units.

The water enters the reservoir through surface inflow and direct precipitation; the water that leaves reservoir comprises of releases through outlets and spillways, evaporation, and losses due to seepage. To the extent possible, all the components of the water balance equation should be independently estimated. The error term δ represents the net effect of errors in the estimation of different components. In practice, it is likely that errors will be present while measuring or computing various terms of the water balance equation. A large value of δ represents a significant error in estimating different variables involved in eq. (10.9). However, a small value of δ does not necessarily indicate that errors are small; may be the errors of opposite sign balance themselves.

The water balance equation may be applied for any time interval. The mean water balance is a term normally used for computations which are spread over an annual cycle, e.g., a water year. Sometimes this term is also used for seasonal water balances. The computations of the mean water balance are simplest in nature; with the shortening of the computational period, a more detailed accounting procedure is required. The additional factors which are to be included in the computations include bank storage during reservoir filling, water loss due to water and ice left on the banks when the reservoir is drawn down, and return of this water to reservoir later on. Sokolov and Chapman (1974) provide a detailed discussion on water balance computations.

10.7 RANGE ANALYSIS

This is an important component of storage analysis. Let x_i, i = 1, 2, ... n, represent a time series of flows at a particular site on a stream. This time series can be a daily, weekly or monthly series. Assume that these flows are being fed into a big reservoir and an amount equal to the mean of the series (x_m) is being taken out. Let

$$S_1 = \Delta x_1 = x_1 - x_m \qquad (10.10)$$

be the increase or decrease (depending on the relative magnitude of x_1 and x_m) in the reservoir content at the end of the first time period. In this manner,

$$S_i = \Delta x_1 + \Delta x_2 ... + \Delta x_i = \sum_{j=1}^{i} \Delta x_i \qquad (10.11)$$

represents such a change at the end of the i^{th} time period. The maximum of n values of S_i, denoted by S_n^+ is called the maximum surplus, or surplus or maximum partial sum of deviates. The minimum of n values of S_i represented by S_n^- is called minimum deficit or deficit or minimum partial sum of deviations. The sum of magnitudes of surplus and deficit

$$R_n = S_n^+ + |S_n^-| \qquad (10.12)$$

is called the *range*. The terms surplus, deficit and range are graphically represented in Fig. 10.7.

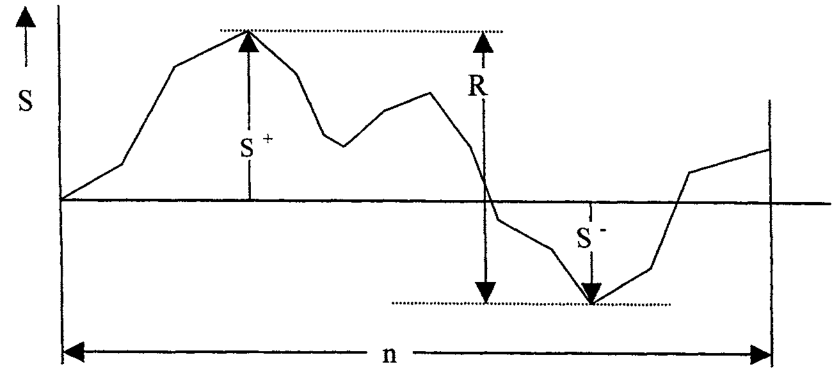


Fig. 10.7 Definition diagram of surplus, deficit, and range.

The statistic range is always greater than zero and represents the storage capacity required in a reservoir to maintain an outflow equal to x_m if the x_i were inflows. However, this implies that there is no loss of water due to spill or other reasons.

The statistic range depends on the properties of the series as well as its length. As the length of the series increases, the range will either increase or remain the same. So far, the outflow from the reservoir is assumed to be equal to the mean of all inflows. Different values of the parameters under discussion are obtained if the outflow is not x_m but is equal to the mean of the subseries of size n. In such cases, the adjective adjusted is used with the parameters and they are called adjusted surplus, adjusted deficit and adjusted range.

10.7.1 Hurst Phenomenon

Extensive investigation on the properties of range were carried by Hurst et al. (1965). It was concluded that the rescaled range, R/σ, where σ is the standard deviation of the data, increases with the length of the series. It can be proved if a series follows a normal distribution and its members are independent of each other then, for large value of n

$$R/\sigma = 0.5\sqrt{n\pi} = 1.25\sqrt{n} \tag{10.13}$$

However, Hurst found that for the natural phenomenon the relationship between R/σ and the length of the series is given by

$$R/\sigma = (n/2)^H \tag{10.14}$$

where H is a variable. This equation was derived based on an analysis of 75 phenomena and 690 portions of these. It was found that the variable H is normally distributed with a mean of 0.73 and a standard deviation of 0.09. Of the above two equations for rescaled range, the eq. (10.11) expresses that it increases with a 0.5 power of n while the eq. (10.12) shows that it increases with a 0.73 power of n. This discrepancy in the value of exponent H is termed as '*Hurst phenomenon*'.

After the discovery of this unusual behavior of natural variables by Hurst, a tremendous amount of research has been conducted to study the causes of this behavior and a number of models have been developed to reproduce this phenomenon in the time series models. Before going for a discussion of the causes of the Hurst phenomenon, the dependence structure of hydrologic time series is discussed.

10.7.2 Dependence in Hydrologic Time-Series

In a dependant time-series, an element is influenced by its predecessors or the past of the series shapes the present. The dependence of a series can be analysed either by correlogram analysis or range analysis. Assume that the hydrologic time series being studied is stationary. Such a series is generated by a stationary process whose probability laws do not change with time. Generally the geophysical, biological and other natural processes are nonstationary but these can be assumed stationary for a relatively short time span.

A hydrologic time series displays two types of dependence: long-term and short-term. It is observed that the autocorrelation coefficient of a hydrologic time series dies out as the lag increases. This implies that a particular value of the variable is influenced only by the recent past values of the series, the distant values do not affect the present. This type of dependence is termed as short-term dependence and the process is said to be a short memory process. In these processes, the incidents tend to fade from the system memory as the time passes. Although this also looks intuitively correct, this observation fails to explain the Hurst phenomenon.

The Hurst phenomenon can be satisfactorily explained by the long-term dependence which implies that the process has infinite memory. The long-term dependence is associated with the failure of the correlogram to die at high lags. However, it is difficult to explain this physically. For example, it is hard to figure out as to how the discharge of a particular day is affected by the discharge of say 100 past days and by what mechanisms the impact of a hydrological event is carried over for years together. These counter arguments have given rise to a controversy about the appropriate explanation of the Hurst phenomenon. The short-term dependence, although physically believable, cannot explain the Hurst phenomenon. On the other hand, the long-term dependence can explain this feature but it cannot be physically explained. Two types of models have been developed in hydrology corresponding to these two types of dependence: the long memory models and the short memory models.

The concept of stationarity is also a pre-requisite for proper interpretation of the Hurst phenomenon since the rescaled range is also a function of deviations from the mean. Klemes (1974) conducted a number of simulations using white noise and the mean level was changed in different ways. It was shown that the value of H increased with this type of nonstationarity. The nonstationarity assumption may not be helpful in practice since it is difficult to fit a nonstationary model to a given hydrologic series.

Another explanation, put forward to explain the Hurst phenomenon, was that the length of the available record is not long enough for H to attain a value of 0.5. It has been argued that if a sufficiently long series of observations is available, H would tend to attain approach 0.5. A plausible explanation of the value of H higher than 0.5 is the persistence, the higher values being the effect of dependence on observed natural series. The dependence can be taken into account in Markovian models but these models cannot reproduce $H > 0.5$. Hence, if the dependence is considered to be the likely cause of the Hurst phenomenon, then long memory models are required. The short-term persistence is caused by the storage effect.

Kumar (1982) pointed that in geophysical processes, the memory manifests itself mostly through the conservation of mass and momentum and it has the Markovian property that the past influences the future only through its influence on the present. Thus, once the present state has been arrived at, it is no longer significant, from the point of view of future development, as to how it was arrived at.

Short memory models have been used in hydrology to generate synthetic data. For many hydrologic studies, particularly those concerned with design and management of water resource systems, sufficiently long data series are required to determine the operating rules and testing the system under various conditions to evaluate their performance. It may be mentioned that a longer generated series does not contain any further information than its parent series. But its use helps with greater extraction of the information already contained in the parent series. The autoregressive models (see Chapter 4) are a class of short memory models which have been extensively used in hydrology.

Salas (1972) has derived relationships for determination of range of periodic-stochastic processes which may be employed to obtain the required storage capacity.

10.7.3 Sensitivity of Reservoir Storage to Inflow Statistics

The minimum reservoir capacity that is required to meet a given demand depends on the generating mechanism of inflows, apart from the nature of demands themselves. Wallis and Matalas (1972) conducted simulation experiments to determine the sensitivity of the reservoir capacity to various parameters of an inflow sequence, including the Hurst exponent H. They used two approaches to generate inflows: the Markov process and the fractional Gaussian noise process. The most important parameter of the Markovian process is ρ_u which is *lag u* serial correlation coefficient. The fractional Gaussian noise model was proposed by Mandelbrot and Wallis (1969).

Wallis and Matalas (1972) generated a number of sequences using these two models and the mean storage for various levels of development was determined using the Sequent Peak Algorithm. It was found that over certain ranges of values of the level of development (α), ρ_u, and H, the mean storage was insensitive to the inflow generating process. For $\alpha < 0.80$, the mean storage depends on ρ_u. For $\alpha \geq 0.80$, the mean storage mainly depends on H. Klemes et al. (1981) conducted simulation experiments to determine differences in reservoir performance reliability when inflows are generated using long memory models and short memory models. The reliability of the reservoir was characterized in three different ways:

- Occurrence-based reliability R_a which is the number of nonfailure years expressed as a percentage of the total number of years in a given period.
- Time-based reliability R_t which is the total duration time of all nonfailure intervals expressed as a percentage of the total length of the given period.
- Quantity-based reliability R_v which is the actual amount of water supplied expressed as a percentage of the total demand during the given period.

Usually, at least some years contain shorter or longer periods of nonfailure and during most failure periods, the outflow is not reduced to zero. This leads to the condition $R_a \le R_t \le R_v$. Based on simulation experiments, the following observations were made by Klemes et al. (1981):

- The short-memory model leads to over-estimation of reservoir performance reliability as compared to the long memory model.
- Overestimation is, in general, highest for annual reliability, lower for time-based reliability and lowest for quantity-based reliability.
- Overestimation of all the three reliability characteristics is very small for reservoirs with storage coefficients up to about one for any draft ratio. For reservoirs with the storage coefficient greater than one, it is small for draft ratios up to about $D = 0.6$ to 0.8 and over $D = 1.1$, depending on the inflow parameters, such as the coefficient of variation and lag-one serial correlation coefficient.
- Overestimation is maximal for draft ratios close to one and increases with reservoir coefficients up to about 2 or 3. For higher values of this coefficient, no further increase was detected.
- Overestimation slightly increases with the variability of inflows and decreases with the increase of the lag 1 correlation coefficient.

Thus, it was concluded that the use of short or long memory model makes no difference in the reservoir performance reliability if the draft ratio is either very low or very high. The length of the memory in the annual inflow series is irrelevant if the regulation itself has no over-year memory. For example, if the draft is low, the reservoir fills up every year and if it is high, the reservoir empties every year; in both cases, only seasonal regulation is involved.

It was further pointed out that the decision makers with high-risk aversion would prefer long-memory models; those with low-risk aversion, short-memory models. The replacement of a short-memory streamflow model with a long-memory model amounts to incorporation of a small safety factor into the reservoir performance reliability. However, in most practical cases, this factor is much smaller than the accuracy with which the performance reliability can be assessed.

10.8 REGULATION REGIME FUNCTION

The storage reservoirs are the most effective way of regulating natural flow of a stream. The reliability of getting a specified amount of release mainly depends on, inter alia, the storage capacity of the reservoir. The more is the storage capacity, the higher is the reliability of supplying a given amount of water or the higher will be the yield for a specified reliability. This is so because the reservoir basically provides the storage space to carry inflows from a period of excess to that of deficit. A study of the storage-reliability-yield relationship is essential to provide preliminary estimates of capacity or yield of a reservoir. The relationship relates inflow characteristics, reservoir capacity, release, and reliability. This analysis is the main aim of the stochastic theory of storage. The methods of storage-yield analysis can be broadly classified into sequential and non-sequential methods. Between two, sequential methods which use historical inflow series, are more popular; the most common example being the mass curve method. The simulation technique is widely used these days.

The inflow process, the reservoir storage, and the outflow process constitute the streamflow regulation system. The operation regime of this system is specified by the storage capacity of the reservoir (S), yield (q), and a measure of reservoir reliability (r). The relationship among these three is designated by a function (Klemes, 1981):

$$\phi = \phi(S, q, r) \tag{10.15}$$

where $S > 0$, $q > 0$, $0 < r \leq 100\%$. This function is called the regulation regime function, or regime function, or storage-yield function, or storage-draft function. Any two of the three variables involved in equation (10.15) can be regarded as independent and the third one following an appropriate functional form:

$$S = S(q, r) \tag{10.16a}$$
$$q = q(S, r) \tag{10.16b}$$
$$r = r(S, q) \tag{10.16c}$$

Klemes (1981) pointed out that the available methods based on the stochastic storage theory can directly handle only eq. (10.16c). This equation can be regarded as the basic equation by means of which the regime function can be evaluated. Eq. (10.16a) and (10.16b) can be solved only for the finite deterministic inflow series and $r = 100\%$ with the aid of the mass curve method. The nature and shape of the regime function depends on the statistical properties of inflows. The shape also depends on the length of the time period used in the analysis. The required storage size reduces with increase in the time period due to the averaging out of fluctuations. The storage capacity arrived at using the mean annual flows is termed as long-term storage and the one using mean monthly (or of durations of the same order) gives a very close estimate of the storage capacity required to meet the given demands. Seasonal or within-year storage is the difference of these two storage values.

10.8.1 Development of Components of Regime Function

As pointed out earlier, an iterative search using simulation is best suited for developing different components of the regime function or determining the value of the third variable knowing the other two. The different components of the regulation regime function for Dharoi Reservoir are presented in graphical form in Fig. 10.8a, b & c, which correspond to eqs. (10.16a), (10.16b), and (10.16c), respectively. The yield is expressed as a ratio of the mean inflow and it is termed as the degree of regulation, level of development, or draft ratio, etc. Similarly, the storage capacity is also expressed as a ratio of the mean annual inflow total and can be termed as the storage ratio or storage coefficient. For the Dharoi reservoir, the mean annual inflow was 86.808×10^7 cubic meter. A distribution of yield among different months was adopted, based on the irrigation demand and an annual reliability measure was adopted. It is readily apparent from Fig. 10.8a that the value of the reservoir yield decreases rapidly with increase in the reliability *r*. Similarly, Fig. 10.8b shows that after a certain limit, the marginal requirement of the storage space for small increases in the storage required are quite large, more so for smaller values of the yield. In other words, it means that smaller degrees of regulation for the reservoir can be obtained with quite small storage ratios.

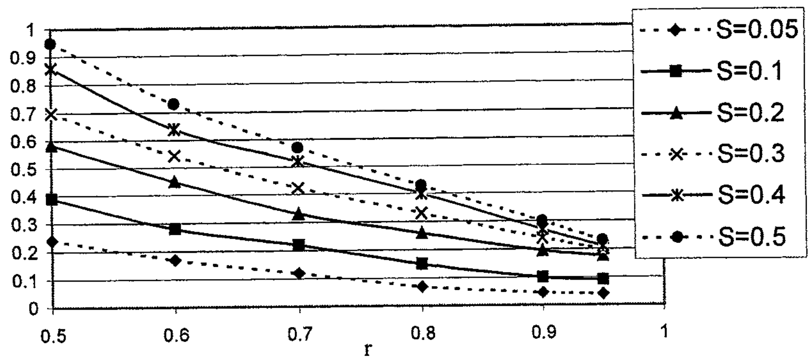


Fig. 10.8a Regulation regime function S= S(q,r).

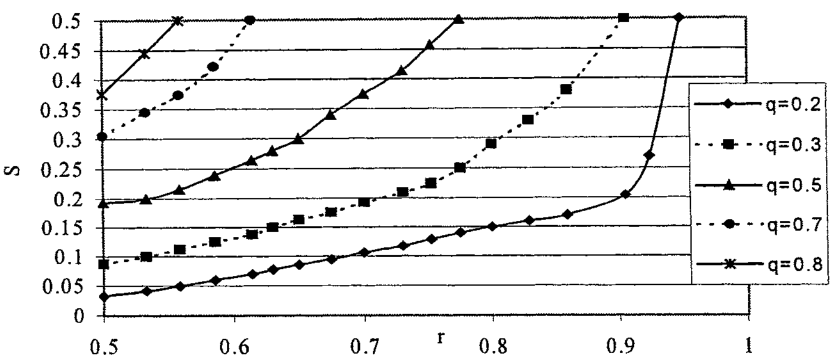


Fig. 10.8b Regulation regime function q= q(S,r).

10.9 RESERVOIR CAPACITY COMPUTATION

Having estimated the water requirements for an intended project and having assessed the available water at a prospective site, a planning engineer is faced with one of the three situations:

a. The rate at which water is available is always in excess of the requirements.

b. The total available water over a period of time is greater than or equal to the overall requirements, but at times, the demand exceeds the availability.

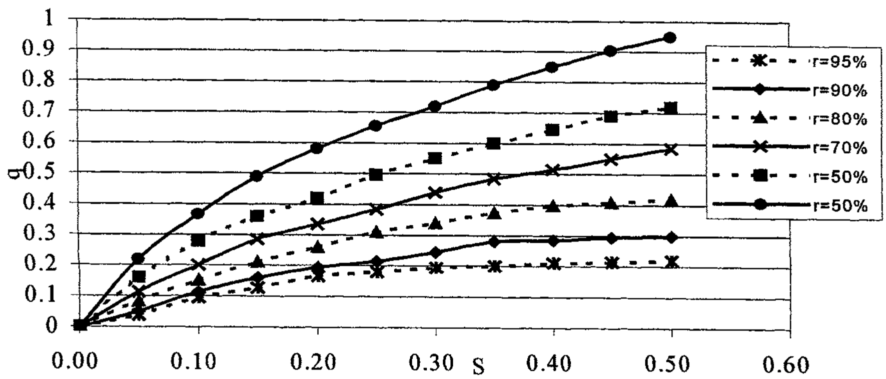


Fig. 10.8c Regulation regime function r= r(S,q).

c. The total available water is much less than the requirements.

In the first case, water can be used directly from the stream as and when needed. A storage reservoir is the solution to the second case. In the third case, a supplemental source or an alternative site has to be explored.

Once it is decided that a storage reservoir is required at a particular site, the next important decision is to finalize the capacity of the reservoir. The required storage to meet given demands depends on three factors: the variability of streamflows, the size of demands and the reliability of meeting the demands. The procedures for estimating the storage capacity needed to meet given demands or the possible yield from a given project design and data constitute the *storage-yield (SY) analysis*. The selection of a suitable method mainly depends on the following factors:

- The level of study: More accurate methods should be selected in the design stage while an approximate method can be adopted in the planning stage.
- Reliability of input data: An accurate method is useful only if the input data are reliable.
- Time and facilities available for study: A sophisticated method can be adopted if sufficient time and facilities are available.

The storage capacity of a reservoir is divided into a number of zones based on the useful purposes the reservoir is required to serve. These zones are discussed below.

10.9.1 Storage Zones in a Reservoir

For ease in analysis and operation (see Section 10.4), the entire reservoir storage space is conceptually divided in a number of zones by drawing imaginary horizontal planes at various elevations (see Fig. 10.1). The lowest zone is the dead (or inactive) zone and its storage capacity is denoted by S_{min}. The bulk of the storage capacity for conservation purposes is provided in the conservation (or active) storage zone and is denoted by S_{active}. The top level of the conservation zone is termed as Full Reservoir Level (FRL) or normal pool level. If the storage space above FRL is exclusively reserved for flood control, the maximum storage capacity is $S_{max} = S_{min} + S_{active}$. In some cases, the flood control space may be temporarily used to store water for use in the dry season. The highest level up to which the water is allowed to rise in a reservoir is known as the Maximum Water Level (MWL). When there is flow over the spillway, a temporary surcharge storage is created above the flood control pool.

Dead Storage Zone

Dead storage is provided in a reservoir to serve two purposes:

a) Most rivers carry sizeable amount of sediment either as suspended or bed load. Upon entering a reservoir, the velocity of flow reduces and hence its carrying capacity is lost. So the sediment settles down and keeps on accumulating. On account of this accumulation, the effective storage capacity of the reservoir goes on reducing with time. This phenomenon is termed as reservoir sedimentation and is discussed in Chapter 12.

b) For efficient working of turbines in a hydropower project, it is necessary that the head variation must be within a specified range and a minimum head must always be available.

The storage provided in dead zone is the greater of the above two factors.

The bulk of the storage space is provided by the conservation zone. The methods for its estimation are discussed next.

10.10 STORAGE REQUIREMENT FOR CONSERVATION PURPOSES

A number of techniques are available to compute the storage capacity for conservation purposes, such as irrigation, municipal and industrial water supply, the hydropower generation. Depending on the type of data and the computational technique used, the reservoir capacity computation procedures are classified into the following categories:

a) Critical Period Techniques

The techniques based on the critical period concepts are the earliest techniques of storage-yield analysis. The critical period is defined as the period in which an initially full reservoir, passing through various states (without spilling), empties. One such method, known as the *Mass Curve Method* was the first rational method proposed to compute the required storage capacity of a reservoir. The other popular method in this group is the *Sequent Peak Method* proposed by Thomas and Bourdon. Some analytical methods, such as Alexander Method and Dincer Method, are also based on the critical period concept, but these are not commonly used and are of academic interest only.

b) Simulation/Optimization Techniques

Among optimization techniques, those based on Linear Programming (LP) and Dynamic

Programming have been found to be particularly suitable. The simulation approach can be used as stand-alone or it can be used to further modify and test the results of critical period or optimization methods.

c) Probability Matrix Methods

These methods use statistical laws to analytically solve the storage-yield problems. Some of the well-known methods in this class are Moran's and Gould's methods. Moran formulated a system of simultaneous equations involving time and water volume. The methods could be applied to any streamflow distribution. However, the solution of the problem was possible under simplified assumptions only and hence the utility of this method for real-life problems is rather limited. These methods are not commonly used for real-life problems. A comparative analysis of selected methods of storage-yield analysis is given in Table 10.2.

Table 10.2 Comparison of selected methods of storage-yield analysis

	Mass Curve	Sequent Peak	Simu-lation	Dincer	Alexander	Moran	Modified Gould
Assumes initially full reservoir	Yes	Yes	Yes	Yes	Yes	No	No
Gives reservoir contents	Yes	Yes	Yes	No	No	No	Yes
Accounts for evaporation	No	No	Yes	No	No	Yes	Yes
Variables demands	No	Yes*	Yes	No	No	No	Yes*
Reliability measures							
R_a	No	No	Yes	No	No	Yes	Yes
R_t	Yes	No	Yes	No	No	Yes	Yes
R_v	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Computational load	Low	High	High	Low	Low	High	High

* Provided annual demand remains constant.

Kottegoda (1980), McMahon and Mein (1986), Klemes (1981), and Nagy et al. (2002) have discussed methods such as the Moran's method, Alexander's method, Gould's method, Phatarford's method, etc. It is clear from Table 10.2 that simulation is the best method for SY analysis.

10.10.1 Mass Curve Method

Also known as the Rippl mass curve method, it is a simplified method commonly used in the planning stage. The method considers the most critical period of recorded flow. In the methods based on the critical period concept, a sequence of streamflows containing a critical period is routed through an initially full reservoir in the presence of specified demands. The reservoir capacity is obtained by finding the maximum difference between cumulative inflows and cumulative releases. Define a function X(t) as:

$$X(t) = \Sigma_t\, x(t)\, dt \tag{10.17}$$

where x(t) may represent monthly flows. The graph of X(t) versus time is known as the mass curve. In the mass curve method, the storage capacity can be determined either graphically or analytically. The method proposed by Rippl in 1883 to determine the storage capacity of a reservoir, is a graphical technique. According to Klemes (1979), Rippl plotted a residual mass curve Z of reservoir inflows relative to draft q:

$$Z_t = \int_0^t (x-q)d\tau = \int_0^t x d\tau - \int_0^t q d\tau = X_t - Q_t \tag{10.18}$$

and this was used to determine the smallest capacity of the reservoir that is necessary to ensure the release at the desired rate without failure throughout the whole period under consideration. This reservoir will be empty only in the most critical period. The concept is illustrated in Fig. 10.9.

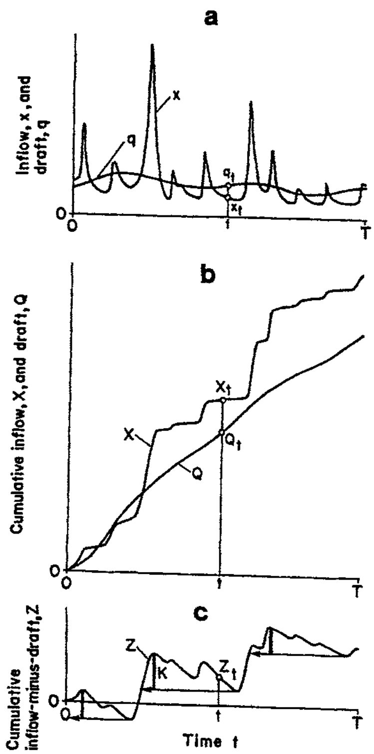


Fig. 10.9 Definition sketch for Rippl's mass curve method: (a) hydrographs of inflow x and draft q, (b) mass curves of inflow X and draft Q, and (c) residual inflow mass curve Z defined in Rippl's sense eq. (10.18) and his procedure of determining the storage capacity K necessary for non-failure reservoir operation during the period T [Source: Klemes (1979)].

The mass inflow curve and mass demand (here demand includes the total water demand and evaporation) curve are accumulated separately. For a constant draft, the yield mass curve is a straight line having a slope equal to the draft rate. At each high point on the mass inflow curve, a line is drawn parallel to the yield curve and extended until it meets the inflow curve. The maximum vertical distance between the parallel yield line and the mass inflow curve represents the required storage. The mass curve of inflows for Dharoi reservoir is plotted in Fig. 10.10. The line AB is the mass curve of demands. Two lines parallel to line AB, namely line CD and EF, are drawn so that they are tangent to the mass curve of inflows at points C and E. The maximum vertical distance between the mass curve of inflows and line CD and EF is noted. The maximum of these is the required storage.

The mass curve technique, although simple and straightforward, is not without shortcomings. This method is suitable when the draft is constant. It is not possible to consider evaporation losses in a meaningful way in this method which could be significant in arid climates. As with all deterministic methods of analysis, the particular set of stream flow figures used is just one sample from a large population and hence conclusions based on that sample will include a sampling error of unknown magnitude. The method has the implicit assumption that the storage which would have been adequate in the past will also be adequate in the future. Although this is not clearly true, the error caused just on this count is not really serious, particularly if sufficiently long flow series has been considered. However, this problem will arise in any other method since the true future is not known and Rippl's method is not tied to the use of the historical record (Klemes, 1979). Some methods address this problem by explicitly considering the stochasticity of inflows.

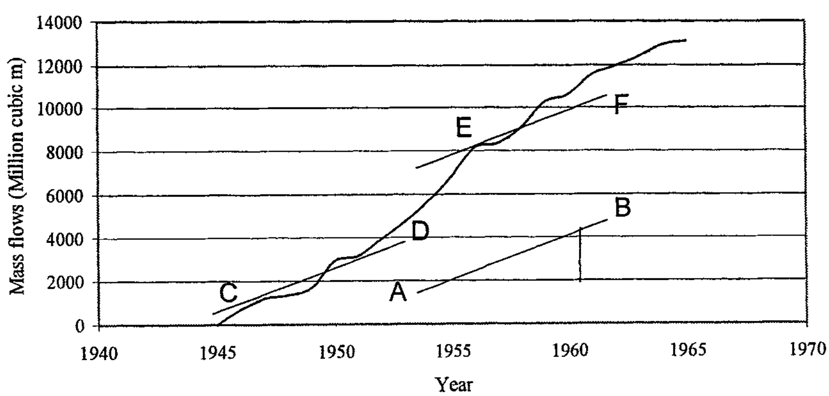


Fig. 10.10 Mass curve for storage analysis.

One more drawback of the mass curve that is quoted in the literature is that an explicit economic analysis cannot be done with this technique. The storage size cannot be related to the economic life of the project and usually an estimate of the storage increase with the increase in the length of the record used. The objective of this analysis is not economic; there is no comparison of costs and benefits when using the mass curve method. Furthermore, the size of storage cannot be computed for a particular level of reliability. Klemes (1979) provided a systems-analytic interpretation of the mass-curve method and noted that it represents a backward-moving, forward-looking recursive maximization when the method is written as

$$K_i = \max(k_{i-1}, C_i), \qquad i = 1, 2, \ldots n \qquad (10.19)$$

where subscript *i* runs backward in time, C_i is the minimal fillup necessary for the i^{th} dry season, and K_i is the minimal storage capacity necessary for the period from the beginning of the i^{th} dry season to the end of the whole period *T*. Klemes (1979) also provided an economic interpretation of the mass curve method. Assuming a convex loss function, the regulation based on a firm value of the target release is optimal under conditions of extreme hydrologic and economic uncertainty about the future. Further, the regulation that is aimed at the greatest possible equalization of outflow is optimal under conditions of perfect knowledge of future streamflow combined with minimum economic uncertainty.

10.10.2 Sequent Peak Algorithm

An analytical solution of the mass curve method is given in the sequent peak algorithm. This method was proposed to circumvent the need to choose the correct starting storage which is required in the mass curve procedure. The computations are quite simple and can be carried out as follows. Let I_t be the inflow to the reservoir in the period *t*, R_t be the release from the reservoir, and S_t the storage at the beginning of *t*. The reservoir is assumed to be empty in the beginning. The mass curve of the cumulative net flow volume (inflow - outflow) against time is used. This curve will have many peaks (local maxima) and troughs (local minima). For any peak P_i the next following the peak of magnitude greater than P_i is called a sequent peak. To take care of the case when the critical period falls at the end of the record, computations are performed for twice the length of the inflow record assuming that the inflows repeat after the end of the first cycle. The variable S_t is calculated as:

$$S_t = \begin{cases} S_{t-1} + R_t - I_t & \text{if positive} \\ 0 & \text{if negative or zero} \end{cases} \qquad (10.20)$$

The required storage capacity is equal to the maximum of S_t values.

Example 10.3: The monthly inflow data for a reservoir are available for 56 months. The required constant release is 34.0 million m^3. Find out the required storage using the sequent peak method.

Solution: Following eq. (10.20), computations are shown in Table 10.3. As seen from the table, the required reservoir capacity would be 675.8 million m^3 which is the maximum of the last column in the above table. Here the calculations need not be repeated for the second cycle because the storage at the end of first cycle is close to zero and therefore the second cycle would be identical to the first cycle.

The sequent peak algorithm can consider variable release from the reservoir. The reliability of the reservoir can be obtained indirectly. Since the reservoir would be able to meet the worst drought from the record, the implied probability of failure would be

1/(N+1). The algorithm is very fast and easy to program. A single historical record is used to compute the storage and hence the method is limited in that sense. It is also not possible to exactly consider the losses, but these can be approximately included in the releases.

Table 10.3 Illustration of sequent peak algorithm (all data in million m^3).

Period (t)	Storage S_{t-1}	Inflow I_t	Release R_t]	Storage S_t
1	0.00	17.10	34.0	16.90
2	16.90	47.20	34.0	3.70
3	3.70	76.70	34.0	0.0
4	0.00	2.60	34.0	31.40
5	31.40	0.70	34.0	64.70
6	64.70	0.0	34.0	98.70
7	98.70	0.0	34.0	132.70
8	132.70	0.0	34.0	166.70
9	166.70	0.60	34.0	20.10
10	20.10	0.10	34.0	234.0
11	234.00	0.70	34.0	267.30
12	267.30	0.0	34.0	301.30
13	301.30	6.20	34.0	329.10
14	329.10	10.10	34.0	353.0
15	353.00	40.7	34.0	346.30
16	346.30	0.6	34.0	379.70
17	379.70	0	34.0	413.70
18	413.70	0	34.0	447.70
19	447.70	0.3	34.0	481.40
20	481.40	0.2	34.0	515.20
21	515.20	0.4	34.0	548.80
22	548.80	0	34.0	582.80
23	582.80	0.3	34.0	616.50
24	616.50	0	34.0	650.50
25	650.50	8.7	34.0	**675.80**
26	675.80	184.9	34.0	524.90
27	524.90	527.2	34.0	31.70
28	31.70	48.1	34.0	17.60
29	17.60	17.10	34.0	34.50
30	34.50	47.20	34.0	21.30
31	21.30	76.70	34.0	0.0
32	0.00	2.60	34.0	31.40
33	31.40	0.70	34.0	64.70
34	64.70	0.0	34.0	98.70
35	98.70	0.0	34.0	132.70
36	132.70	0.0	34.0	166.70
37	166.70	0.60	34.0	20.10
38	20.10	0.10	34.0	234.0
39	234.00	0.70	34.0	267.30
40	267.30	0.0	34.0	301.30
41	301.30	6.20	34.0	329.10
42	329.10	10.10	34.0	353.0
43	353.00	40.7	34.0	346.30
44	346.30	0.6	34.0	379.70
45	379.70	0	34.0	413.70
46	413.70	0	34.0	447.70
47	447.70	0.3	34.0	481.40
48	481.40	0.2	34.0	515.20
49	515.20	0.4	34.0	548.80
50	548.80	0	34.0	582.80
51	582.80	0.3	34.0	616.50
52	616.50	0	34.0	650.50
53	650.50	8.7	34.0	675.80
54	675.80	184.9	34.0	524.90
55	524.90	527.2	34.0	31.70
56	31.70	48.1	34.0	17.60

10.10.3 Stretched – Thread Rule

One of the objectives of streamflow regulation problems with the aid of the mass curve analysis is to determine regulation aimed at the greatest possible equalization of the reservoir outflow (Klemes 1981). The need for this regulation arises from a desire to reduce the losses that could arise due to flows being either too high or too low. Such regulation can be considered to have the maximum economic effect and has also been termed as ideal. A simple method to determine such a regulation policy was introduced in Europe in 1923 by Varlet and is known as *The Stretched - Thread Rule* (Klemes 1979).

In this method, the mass curve of inflows is drawn as shown in Fig. 10.11. Now this mass curve is shifted upwards by a distance equal to the reservoir storage capacity. Let a thread be stretched between these two mass curves whose lower end is either midway between the two curves or zero; the upper end is also fixed the same way. The shape of this thread represents the shortest path between two opposite ends of the corridor formed by two mass curves. This thread is the mass curve of the reservoir outflow which gives the greatest possible equalization of outflow. The lower mass curve represents the line of full reservoir and the upper, the line of empty reservoir. The volume of water in storage at any time is given by the vertical distance between the stretched-thread and the upper mass curve.

Klemes (1979) detailed a procedure for numerical computation of shortest path when the corridor is enclosed within two broken lines consisting of straight-line segments and the end-points of the shortest path are specified. The lower broken line is given as a series $\{X_t^*\}$, t = 0, 1, ..., N, the upper broken line as $\{X_t^{**} = X_t^* + K_t\}$, where K_t is the width of the corridor so that $K_t > 0$ for all t. In the storage reservoir problem, a variable width of the corridor arises if constraints on storage additional to those of empty and full reservoir are specified, for instance variable requirements on freeboard and minimum storage level throughout the year. The steps suggested are as follows:

1. A straight line connecting the end points A_o and A_N is computed.
2. The corridor boundaries are checked to see whether any of them are crossed by the line A_oA_N. If no crossing is recorded, the line A_oA_N is the desired shortest path. If X^* crosses A_oA_N, the point of maximum distance of X^* above A_oA_N is identified as a corner point A_t of the shortest path; if X^{**} crosses A_oA_N, the point of maximum distance of X^{**} below A_oA_N is identified as another corner point A_j.
3. The corner point closest to the starting point A_o, in this case A_j, is regarded as an end point of the shortest path in the period (0,j).
4. Steps 1-3 are repeated with A_j replacing A_N.
5. If no additional corner points are identified in the interval (0, j), the straight line A_oA_j is the first segment of the shortest path and the search moves to the next interval (j, i) with A_j and A_i representing the starting and the end points, respectively. If, however, an additional corner point, say A_k, is identified in the period (0, j), the search moves to the interval (0, k).
6. In general, the search always moves forward in time only after the shortest path in the whole past period has been found.

Fig. 10.11 shows the stretched-thread diagram for the Dharoi reservoir.

10.10.4 Storage-Yield Analysis

The storage yield (SY) analysis is carried out to determine the smallest volume of storage required to meet given demands with a stated reliability. It is also used to reassess the water demand which can be satisfied by an existing reservoir. The required storage depends on the volume of demand, reliability of meeting them, and reservoir inflows. The SY analysis problems can be of two types: given the storage, calculate the yield; or given the yield, calculate storage. Both optimization and simulation techniques are used in SY analysis.

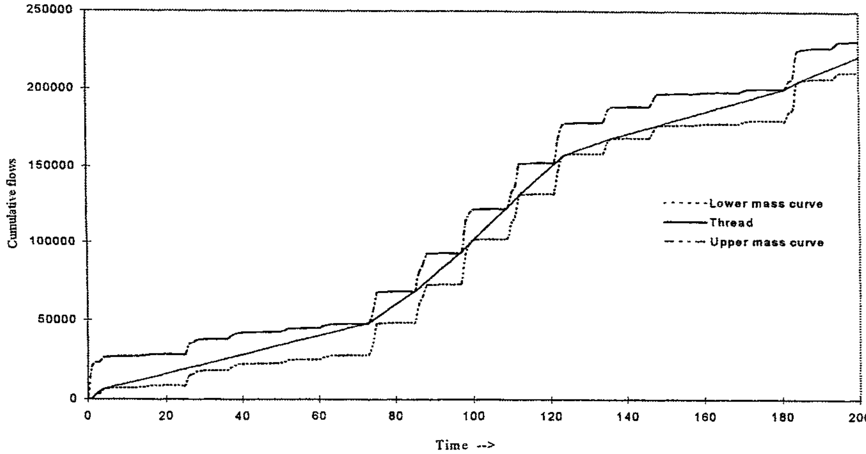


Fig. 10.11 Stretched-thread analysis for Dharoi reservoir.

Optimization Techniques

Among the various available optimization techniques, linear programming (LP) and dynamic programming (DP) have been extensively used for storage-yield analysis. Here, an LP based formulation is discussed. The problem formulation is essentially the same for DP.

Consider that a reservoir is to be constructed at a particular site. Monthly inflow data for the past n months are available. The projected demand of water during a critical year is known along with its distribution among each month. The losses from the reservoir are neglected for the time being. The problem is to find out the minimum capacity of the reservoir which will supply the required quantity of water without failure. Let D be the annual water demand from the reservoir and α_i, $i = 1,2,...12$ be its fractions for different months. Hence, the demand in a particular month will be α_i D. Let I_t be the inflow to the reservoir during the t^{th} month and R_t be the water actually released from the reservoir. The objective is to find the minimum capacity of the reservoir which can meet the demand.

$$\text{Min C} \tag{10.21}$$

Representing the storage content of the reservoir at the beginning of month t by S_t, the continuity equation (neglecting losses) is:

$$S_t + I_t - R_t = S_{t+1} \quad t = 1, 2...n \tag{10.22}$$

This equation has to be satisfied for each of the n months and hence there will be n such constraints in the formulation. The value of S_1 is given as input.

It is also required that the amount of water actually released from the reservoir must be more than or equal to the amount demanded. This can be mathematically expressed as:

$$R_t \geq \alpha_i D, \qquad t = 1, 2 \ldots n \tag{10.23}$$

The index $i = 1, 2, \ldots 12$ represents the calendar month. Since this constraint also must hold for each month, there will be n such constraints. If the capacity of the required reservoir is C, then in any month, from a physical point of view, the storage content of the reservoir must be equal to or less than this value. Hence,

$$S_t \leq C, \qquad t = 1, 2 \ldots n \tag{10.24}$$

Besides the storage S_t, capacity C, annual demand D, and release R_t can take on only positive values. This completes the problem formulation. The problem is quite easy to solve particularly due to the availability of standard package programs.

The computations for firm power can also be done in a similar manner. The maximum firm power output can be obtained by

$$\text{Max}\,[\min HE_t], \quad t = 1, 2, \ldots n \tag{10.25}$$

where HE_t is the amount of hydropower produced during period t. The maximum possible firm power which can be generated depends on the site conditions, hydrology of the area and the capacity of generating equipment. The lower bound of firm power is zero.

10.10.5 Simulation Method (Behavior Analysis)

Simulation is essentially a search procedure. It is one of the most widely used techniques to solve a large variety of problems associated with the design and operation of water resources systems. The reason is that this approach can be realistically and conveniently used to examine and evaluate the performance of a set of alternative options available. Furthermore, serial correlation of inflows, seasonality, etc., are easy to account for. Also, it is easy to present the technique and its results to non-technical persons.

Assume that a site has been identified for construction of a dam. The reservoir has to cater for irrigation for a nearby area and the target demand of water for different months is given. The problem is to find the required capacity of the reservoir. The elevation-area-capacity table for the site is available. A sufficiently long series of streamflows at the site is available and the reliability to meet the demands has been specified.

The computational steps using a binary search method are as follows:

a) At the beginning of computations, the upper bound of the storage capacity is decided based on the water availability, site conditions, etc. The lower bound of storage is taken as dead storage S_{min}.

b) The reservoir is initially assumed to be full. The effect of the initial storage value will not be significant if the operation of the reservoir was simulated using a long inflow series.

c) A trial value of storage as the average of upper and lower bounds is assumed.

d) The operation of the reservoir is simulated for the whole period of record. At each time interval t, an attempt is made to satisfy the demand to the extent possible. If there is not enough water in the reservoir to meet the demand during any period, the demand is met to the extent possible and the period is treated as failure. If the available water in the reservoir in a period is less than S_{min}, no release is made. The storage is depleted by evaporation only and the reservoir is assumed to have failed during that period. If there is so much water during the period that there is no place to keep it even after meeting all the demands, the extra water over the storage capacity is spilled. The storage at the end of the period is computed using the continuity equation

$$S_{t+1} = S_t + I_t - E_t - R_t \tag{10.26}$$

Step (d) is repeated for all the n periods.

e) Now, the reliability of the reservoir (REL) is computed by

$$REL = 1.0 - FAIL/n \tag{10.27}$$

where FAIL is the number of failures (number of periods when release R_t < demand D_t).

f) If this reliability is less than the desired value, it means that the trial value of the reservoir capacity is small. Hence, the present value is adopted as the lower bound for the next iteration. The feasible region below this lower bound is discarded and the trial value for the next iteration is chosen midway the upper bound and new lower bound. Go to step (d).

g) If, however, the reliability comes out to be higher than the requirement, the trial size of the reservoir is bigger than what is necessary and hence the region between the current value and the upper bound is discarded for further examination. The present capacity value becomes the new upper bound. Again, the trial value for the next iteration is chosen as mean of new upper bound and old lower bound. Go to step (d).

h) The iterations are performed till the desired value of the reliability is achieved.

This method converges quite rapidly as the feasible region is halved every time. It may be seen that in this method, generation of hydroelectric power can also be easily considered. The evaporation losses can be easily considered if the information about the depth of evaporation is available. The evaporation loss E_t is a function of both S_t and S_{t+1}.

Undoubtedly, a sufficiently long and reliable inflow data series is the prime requirement for SY analysis. Although the method to be adopted for a particular problem will depend on the available data, simulation has been rated as the best method for SY analysis. McMahon and Mein (1986) recommend that the results of the preliminary analyses should be further refined using the behavior analysis.

Many times the available inflow data length is less than desirable or the data may have gaps. In these situations, one has to resort to synthetic generation of stream flow sequences. The purpose of synthetic generation is to have a number of streamflow sequences which are equally likely to occur in the future. A large number of statistical techniques are available for synthetic data generation; many software packages are also available. After the synthetic sequences are available, the operation of the system is simulated using these different sequences. Naturally, a range of storage values will be obtained and this variation indicates the sampling error that is associated with use of the short period data. As the length of the sequences increases, it will be noticed that the variation of the storage value will reduce. The estimate of the required storage increases with the length of the sequence.

The computations to arrive at the preliminary assessment of the live storage for the Dharoi reservoir are given in Appendix 10.B.

10.10.6 Reservoir Screening

Reservoir screening is meant to select the reservoir(s) that should be incorporated into development schemes. Each reservoir with a certain storage capacity is capable of giving a certain yield and therefore, a relation between the storage capacity and yield can be developed. Since there is a relation between storage capacity and cost, one can develop a relationship between cost and yield:

$$C = f(Y) \tag{10.28}$$

where C is cost and Y is yield. If the total yield of the system is to be Y_T, the purpose of reservoir screening is to identify the combination of reservoir(s) which provides this yield Y_T at the minimum cost.

The dynamic programming is a powerful tool to solve this problem. The screening process using a DP model involves two stages. In the first stage, the yield from a reservoir is estimated for various values of storage capacities by simulating each reservoir individually. These results are used in the second stage where selection of reservoirs and their sizes is finalized by DP. This method has two advantages. First, it can use any number of years of input data at the simulation stage, and second it can handle non-linear functions easily. An example of application of DP for reservoir screening was presented in Chapter 5.

10.11 FLOOD CONTROL STORAGE CAPACITY

The requirement of storage space for flood control is in conflict with the requirements for conservation needs. The conservation requirements, such as water supply and hydropower generation, require the storage space to be full while the flood control aspect requires the availability of empty storage space.

From the point of view of analysis, the demands for water supply and hydroelectric power are relatively deterministic in nature, while the demand for flood control storage is completely stochastic. Furthermore, the time period for analysis is usually of the order of one month for conservation purposes while for flood control purposes, it is of the order of a few hours. The requirement of storage space for flood control is estimated by using the design flood hydrograph. An initial reservoir level is assumed at which this flood hydrograph impinges the reservoir. The maximum level attained by the reservoir is computed by routing the hydrograph through the reservoir. The maximum height of the dam is obtained after adding the free board to this level. To begin with, the top of the conservation pool (FRL) is a good choice for initial storage for computations. Before discussing the steps to determine flood control space, it is useful to briefly describe design flood for a reservoir.

10.11.1 Reservoir Design Flood

A design flood is a hypothetical flood (peak discharge or hydrograph) adopted as the basis in engineering design of project components. Some of the common purposes are:

i) Design floods adopted for the safety of structures against failure by overtopping, etc. during floods. For example, the design flood adopted for dams to decide the spillway capacity.

ii) Design floods adopted for flood control and drainage works to provide safety to downstream areas against flooding.

Since the design flood adopted often marks the difference between safety and disaster, utmost attention has been given the world over to select and estimate the design flood that is most appropriate for a given case. Economic, social, and other non-hydrologic considerations influence the philosophy of protection, and hence the selection. Policies have been laid down by most organisations for various applications and are followed unless there are compelling local factors for deviation in the particular case.

Many approaches are used to estimate design floods. The rational formula was widely used in early times and is still used in some countries. The other popular approach is based on the unit hydrograph. The design storm for the project is determined using meteorological data and then the unit hydrograph is applied to determine the corresponding flood hydrograph. When a sufficiently long flow series is available, frequency analysis is carried out to determine the floods of various frequencies. For important projects, the results of various approaches are compared to obtain the design flood.

Design Flood to Determine Spillway Capacity

The design criteria for flood control schemes have evolved over the years, pooling the experiences and practices followed by various organizations and individuals. Law (1992) provided an overview of spillway design flood standards and freeboard requirements in Europe. The prevalent hydrologic design criteria for determining the spillway capacity for India have been detailed in Indian Standard IS 11223-1985, "Guidelines for fixing Spillway Capacity". According to these guidelines the inflow design floods that need to be considered for various functions of spillways are:

(a) Inflow design flood for dam safety

It is the flood for which the dam should be safe against overtopping and structural failure. The criteria for classification of dams are based on size and hydraulic head (see Section 10.1.4). There is no single universally accepted criterion, but a general consensus is to adopt PMF as the design flood for large dams which have high hazard potential. According to the guidelines being followed in India, the following types of spillway design flood are recommended for various sizes of dams.

Table 10.4 Inflow design flood for various types of dams.

Type of Dam	Hydraulic head (m)	Storage Capacity (million m^3)	Inflow design flood
Small	0.5 - 10	7.5 - 12	10 year
Intermediate	10 - 60	12 - 30	SPF
Large	> 60	> 30	PMF

For minor structures, a flood of 50 or 10-year frequency is adopted depending on the importance of the structure. Floods of larger or smaller magnitudes may be used if the hazard involved is high or low, respectively. The relevant parameters to be considered in judging the hazard in addition to the size would be:

i) distance and location of the downstream areas. Due consideration is given to the likely future developments, and
ii) the maximum carrying capacity of the downstream channel at a level at which catastrophic damage is not expected.

(b) Inflow design flood for efficient operation of energy dissipation works

The energy dissipation arrangements for the spillway may be designed for the best efficiency for a smaller inflow flood than the inflow design flood for the safety of the dam.

(c) Inflow design flood to check the extent of upstream submergence

The inflow design flood to check the extent of the upstream submergence depends on local conditions and the type of property and the effects of its submergence. Except for very important structures in the upstream like power houses, mines, etc. for which the levels corresponding to SPF or PMF may be used, smaller design floods and levels attained under these may suffice. In general, a 25-year return period flood for land acquisition and a 50-year return period flood for the built-up property acquisition may be adopted.

(d) Inflow design flood for the extent of downstream damage in the valley

The inflow design flood to check the extent of downstream damage depends on local conditions, the type of property and the effects of its submergence. For important facilities like power houses, the outflows under the inflow design flood for safety of dams and all gates operating conditions are relevant. Normally, the discharge relevant to check the acceptability of the downstream submergence may be smaller than that for power houses at or near the toe of the dam.

For important projects, dambreak studies may be undertaken as an aid to terminate the design flood. Where the professional judgment or studies indicate an imminent danger to present or future human settlements, the PMF should be used as the design flood. Besides the available guidelines, location-specific factors should also be considered while choosing a particular type of flood.

e) Design flood for fixing freeboard

The design of spillways and the size of flood control pool is determined using reservoir routing.

10.12 RESERVOIR ROUTING

The passage of a flood hydrograph through a reservoir is an unsteady flow phenomenon. The routing of a flood wave through a reservoir is known as reservoir routing. It is an important part of the reservoir analysis whose major applications are: fixing maximum water level during reservoir design, design of spillway and outlet works and dam-break flood wave analysis. A reservoir can be either controlled or uncontrolled. The controlled reservoirs have a spillway with gates to control the outflow. The spillway of an uncontrolled reservoir does not have gates.

The continuity equation is the governing equation in all hydrologic routing methods. It essentially states that the difference between the inflow and outflow is equal to the rate of change of storage:

$$I - Q = dS/dt \qquad (10.29)$$

where I is inflow, Q is outflow, S is storage, and t is time.

Over a small time interval Δt, the difference between the total inflow and outflow volumes is equal to the change in storage. Hence, eq. (10.29) can be written as:

$$I_m \Delta t - Q_m \Delta t = \Delta S \qquad (10.30)$$

where I_m, Q_m, and ΔS denote the average inflow, the average outflow and the change in storage during time period Δt, respectively.

For the sake of clarity, it is necessary to introduce the frequently used terms, viz., translation and attenuation characteristics of the flood wave propagation. The translation is the time difference between the occurrence of inflow and outflow peak discharges, and attenuation is the difference between the inflow and outflow peak discharges. Having known the peak discharge of the outflow hydrograph, the attenuation is

Attenuation = [Inflow peak discharge] – [Outflow peak discharge] (10.31)

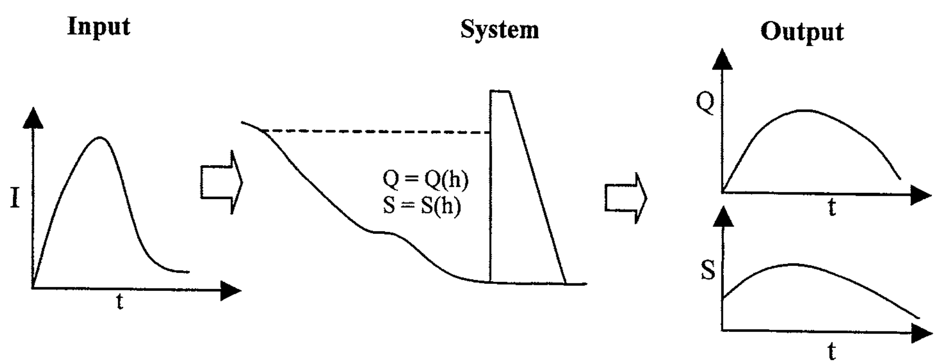
and Translation = [Time-to-peak of inflow] – [Time-to-peak of outflow] (10.32)

10.12.1 Reservoir Routing Techniques

Reservoir routing requires the relationship between the reservoir elevation, storage and discharge to be known. This relationship is a function of the topography of reservoir site and the characteristics of the outlet facility. Using the basic eq. (10.29), several methods for routing a flood wave through a reservoir have been developed, namely:

The Mass Curve Method,
The Modified Puls Method,
The Goodrich Method,
The Coefficient Method.
The Puls Method,
The Wisler-Brater Method,
The Steinberg Method, and

A schematic depiction of reservoir routing is given in Fig. 10.12. Some of these methods are described in the following. Singh (1988) has discussed these methods in detail.



Legend I: Inflow, Q: Outflow, S: Storage, h: elevation, t: time.

Fig. 10.12 Schematic representation of reservoir routing.

10.12.2 Mass Curve Method

This is one of the most versatile methods of reservoir routing, various versions of which include: (i) direct, (ii) trial and error, and (iii) graphical. Here the trial and error version is described.

For solution by trial and error method, eq. (10.29) can be rewritten as:

$$M_{t+\Delta t} - (V_t + Q_m \Delta t) = S_{t+\Delta t} \qquad (10.33)$$

where $M_{t+\Delta t}$ is the accumulated mass inflow at time t+Δt, and V_t is the accumulated mass outflow at *t*. A storage-discharge relationship and the mass curve of inflow should be prepared before obtaining the trial and error solution. The time step size Δt is chosen and the mass inflow hydrograph is computed. The steps of the trial and error solution are as follows:

a) For the current time period, the mass outflow is assumed. It may be a function of the accumulated mass inflow.
b) The reservoir storage is computed by deducting mass outflow from mass inflow.
c) For this storage, compute reservoir elevation and then corresponding outflow.
d) Using this outflow and the outflow for the previous period, compute average outflow and then new mass outflow.
e) Compare the mass outflow of step (d) with that of step (a). If they are not within a desired proximity, another mass outflow is assumed and steps (a) to (d) are repeated.
f) If the two values agree, increment the time step and go to step (a).

10.12.3 Modified Puls Method

This method is also referred to as the Storage-Indication method. The basic law used in the Modified Puls method states: *The inflow minus outflow is equal to the rate of change in storage*. Assuming $I_m = (I_1 + I_2)/2$, $Q_m = (Q_1 + Q_2)/2$ and $\Delta S = S_2 - S_1$, eq. (10.29) is written as:

$$(I_1 + I_2)\,\Delta t/2 - (Q_1 + Q_2)\,\Delta t/2 = S_2 - S_1 \quad (10.34)$$

where suffixes 1 and 2 denote the beginning and the end of the time interval Δt, and *Q* may incorporate the controlled discharge as well as uncontrolled discharge. Separating the known quantities from the unknown ones and rearranging:

$$(I_1 + I_2) + (2S_1/\Delta t - Q_1) = (2S_2/\Delta t + Q_2) \quad (10.35)$$

Here, the known quantities are I_1 (inflow at time 1), I_2 (inflow at time 2), Q_1 (outflow at time 1), and S_1 (storage in the reservoir at time 1). The unknowns are S_2 and Q_2. Since one equation with two unknowns cannot be solved, a relation between storage, *S*, and outflow, *Q* is needed. As the outflow from the reservoir during floods takes place through the spillway, the discharge passing through the spillway can be related with the reservoir elevation which, in turn, can be related to the reservoir storage. Such curves are invariably available for any reservoir. The outflow through spillway can be computed from the following equation:

$$Q = C_d\, L\, H^{1.5} \quad (10.36)$$

where *Q* is the outflow discharge (cumec); C_d is the coefficient of discharge; *L* is the length of spillway (m); and *H* is the depth of flow above the spillway crest (m). Thus, a curve/table of elevation vs. discharge can be prepared. The reservoir storage also depends on the elevation. Therefore, we have

$$S = S(Y) \qquad \text{and} \qquad Q = Q(Y) \tag{10.37}$$

where Y represents the water surface level. The right side of eq. (10.35) can be written as:

$$2S/\Delta t + Q = f(Y) \tag{10.38}$$

Before developing the $[(2S/\Delta t) + Q]$ vs. outflow relation, it is necessary to select a time interval Δt such that the actual non-linear shape of the inflow hydrograph, particularly the crest segment, can be closely linearized within this interval. For smoothly rising hydrographs, a minimum value of $t_p/\Delta t = 5$ is recommended, where t_p is the time to peak of the inflow hydrograph. A computer-based calculation would normally use a much greater ratio, say 10 to 20. Once Δt is fixed, the relations of eq. (10.37) can be used to prepare curves or tables of $(2S/\Delta t \pm Q)$ versus Q since for a given elevation, Q and S are known.

The computations are performed as follows. At the beginning, the initial storage and outflow discharge are known. In eq. (10.35) all the terms in the left hand side are known at the beginning of the time step Δt. Hence the value of $(S_2 + Q_2\Delta t/2)$ at the end of the time step is calculated by eq. (10.35). The outflow can be calculated using the relation between $(S_2 + Q_2\Delta t/2)$ and Q. This procedure is repeated to cover the full inflow hydrograph.

10.12.4 Coefficient Method

In the coefficient method, the reservoir is represented by a single conceptual storage element assuming storage S to be directly proportional to outflow Q:

$$S = K\,Q \tag{10.39}$$

where K is a proportionality factor equal to the reciprocal of the slope of the storage curve that can be a constant or a variable function of outflow. If K is constant, then the reservoir is linear, otherwise the reservoir is non-linear.

For flood routing, a finite difference approximation is normally employed. Equations (10.35) and (10.39) can be combined and written as:

$$\Delta t\,(I_1 + I_2)/2 - (Q_1 + Q_2)\,\Delta t/2 = K(Q_2 - Q_1)$$

or

$$Q_2 = Q_1 + C(I_1 - Q_1) + C(I_2 - I_1)/2 \tag{10.40}$$

where

$$C = \Delta t/(K + \Delta t/2) \tag{10.41}$$

If K is variable, then C can be derived and plotted as a function of Q. For each routing period, the appropriate value of C must be obtained corresponding to the outflow under consideration. Then, routing can be performed by using eq. (10.40).

10.12.5 Reservoir Routing with Controlled Outflow

Most of the big dams have gated spillways and the gates are raised or lowered to control the reservoir outflow. The dam may also have undersluices to release water control for irrigation, water supply, etc. The operation of the spillway gates and undersluices depends on the state of the reservoir, level of demands, and the operation policy. In gated dams, reservoir outflow can be either a) controlled, b) uncontrolled, and c) partly controlled. Incorporating controlled outflow, the continuity equation can be written as:

$$(I_1 + I_2)/2 - (Q_1 + Q_2)/2 - Q_c = (S_2 - S_1)/\Delta t \tag{10.42}$$

where Q_c is the mean controlled outflow from the reservoir during the time interval Δt. Rearranging the terms, eq. (10.42) can be written as:

$$2S_2/\Delta t + Q_2 = I_1 + I_2 + 2S_1/\Delta t - Q_1 - 2Q_c \tag{10.43}$$

When the controlled outflow Q_c is known, the solution can be obtained on the same lines as the modified Puls method. The solution of the eq. (10.43) is simple if the entire outflow is controlled.

10.12.6 Major Applications of Storage Routing

The storage routing has numerous applications. The major ones are discussed below.

Determination of Capacity of Flood Control Pool

The storage routing of floods entering a reservoir is employed to determine the maximum levels attained for given reservoir characteristics, operation policy, and initial conditions. This data is used to demarcate the area likely to be submerged and to fix the heights of the dam. The routing of flood waves of various return periods provides inputs for economic and risk analyses of flood control storage capacity; it also helps in developing a policy for operation of flood control pool.

Sizing Capacities of Outlet Structures

The larger the outlet capacities, the smaller will be the maximum reservoir level for a given shape of the incoming flood and the reservoir level at the beginning of this flood. The storage routing yields the maximum reservoir level attained for the given design of outlet works. The decision variables of spillway design include the type, width, height, and number of spillway openings (either controlled by gates or uncontrolled); the shape of spillway crest; and properties of gates. Optimal design of outlet structures can be obtained by systematically changing the relevant parameters.

Rates of Change of Reservoir Levels

The stability of banks along the reservoir shores, wave erosion, and landslides into the reservoir are important in reservoir management. An important variable that affects these is the permissible rate of change of reservoir water level. This rate depends on soil and rock properties and is expressed in meters/day or cm/hour. The maximum permissible rate of change is used in storage routing for design of outlets and fix the maximum water level.

Effects of Reservoirs on Downstream Floods

The extent of flooding in river reaches downstream of a reservoir can be assessed by knowing the outflow from the reservoir. Storage routing provides the requisite information along with the probability estimates.

Computation of Dam Breach Outflows

To study the dam breach problems, such as the dynamics of breach openings, outflow hydrographs through breaches, etc., storage routing is employed. The outflow hydrograph resulting due to a dambreak and the extent of flooding are input in preparing development plans for areas downstream of a reservoir.

10.12.7 General Comments

The selection of a proper routing time interval Δt is important in all reservoir routing problems. Its value should be neither too long nor too short. If it is too long, the variability of the inflow hydrograph, particularly the crest segment, may not be properly accounted for. If it is too short, it takes more efforts to perform flood routing. Further, Δt is assumed so that the inflow and outflow are approximately linear during this period. Usually, Δt should be one-third to one-half of the travel time through the reservoir. Furthermore, the routing interval Δt need not be constant. It can be kept large when there are small changes in inflow and small when there are large changes therein.

The routing operation performed by the trial and error solution of the mass curve method is simple and easily done. This can be efficiently adapted to complex routing problems. The Puls method and the modified Puls method both have two shortcomings. First, the assumption that the outflow begins at the same time as the inflow implies that the inflow passes through the reservoir instantaneously regardless of its length. Second, it is difficult to choose an appropriate Δt since the negative outflow occurs during recession whenever $\Delta t > 2S_2/Q_2$ or $Q_2/2 > S_2/\Delta t$. The former drawback is not serious if the ratio of $T_t/T_m \leq 0.5$, where T_m denotes the time to peak of the inflow hydrograph and T_t denotes the travel time. T_t is defined as L/u, with L being the length of the reach and u being the average steady state velocity. The latter weakness can be circumvented by plotting discharge versus $[(2S/\Delta t)+Q]$ curve on a log-log paper and comparing the plot with the line of equal values. If the plotted values lie above the line of equal values, the drawn figure must be abandoned and a new value of Δt must be selected. Further negative outflow can be avoided usually by taking Δt less than T_t.

The Wisler-Brater method requires an observed basis for routing computations and so this can best be simulated in controlled conditions. Hence, its use is less for practical purposes. The Steinberg method requires K-curves and their superimposition over storage curves, and thereby involves graphical work before actual routing is carried out.

To solve a reservoir routing problem, the following data are needed:

(a) Water surface elevation vs. storage volume and elevation vs. outflow discharge curve/table,

(b) inflow hydrograph;

(c) initial values of storage, inflow and outflow, and

(d) for the coefficient method, the value of proportionality constant K, which is the reciprocal of the slope of the storage curve, is also needed.

Example 10.4 The elevation-capacity-spillway release capacity for the Dharoi reservoir is given in Table 10.5. The ordinates of hydrograph for a major flood are available at 2-hr interval. Route the flood using the Modified Puls method assuming the initial reservoir elevation at 180.0 m.

Solution: The input data are given in the first four columns of Table 10.6. Using these, the last two columns can be easily computed. The results of computation for the initial reservoir elevation of 180.0 m are shown in Table 10.6. It can be seen from this table that the peak of the outflow hydrograph was 18809 m^3/s while the peak of the inflow hydrograph was 27180 m^3/s. The peak of inflow occurred at 49 hours while for outflow, it was at 57 hours. The maximum water level attained by the reservoir was 192.18m.

Table 10.5 Elevation-capacity-spillway release capacity for Dharoi dam.

S.N.	Elevation (m)	Storage S (10^6 m^3)	Spillway release capacity Q (m^3/s)	[(2S/Δt)+Q] (m^3/s)	[(2S/Δt)-Q] (m^3/s)
1	170.69	29.078	0.0	8077.22	8077.22
2	173.74	58.898	0.0	16360.56	16360.56
3	176.78	103.203	0.0	28667.50	28667.50
4	178.92	157.178	0.0	43660.56	43660.56
5	179.83	180.844	279.23	50513.67	49955.21
6	182.88	304.596	2704.73	87314.73	81905.27
7	185.93	497.225	6718.83	144836.89	131399.23
8	188.98	763.135	12022.37	224004.31	199959.57
9	189.59	829.415	13225.70	243618.76	217167.36
10	190.50	926.847	15095.94	272553.44	242361.56
11	192.02	1108.144	18427.08	326244.86	289390.70
12	193.55	1309.163	21982.90	385639.29	341673.49
13	194.00	1420.000	23400.00	417844.44	371044.44

The reservoir inflow and outflow are plotted in Fig. 10.13. One can observe in eq. (10.29) that when $dS/dt = 0, I = Q$. Thus the peak of outflow hydrograph will fall on the falling limb of inflow hydrograph, as seen in Fig. 10.13. The variation of reservoir water level is given in Fig. 10.14.

Table 10.6 Flood routing through Dharoi reservoir using the Modified Puls method.

Time (Hours)	Reservoir Elevation (m)	Inflow (m^3/s)	Reservoir Storage (10^6 m^3)	Outflow (m^3/s)
1	180.00	566.41	187.74	414.42
3	180.05	854.72	189.73	453.46
5	180.18	1598.98	194.93	555.40
7	180.43	2514.02	205.03	753.27
9	180.79	3438.12	219.98	1046.28
11	181.27	4337.86	239.09	1420.86
13	181.82	5189.46	261.57	1861.50
15	182.44	6000.57	286.68	2353.64
17	183.02	6746.53	313.68	2894.04
19	183.47	7414.61	341.72	3478.34
21	183.92	7981.87	369.98	4067.30
23	184.36	8452.56	397.78	4646.49
25	184.78	8848.77	424.60	5205.34
27	185.18	9162.28	450.05	5735.73
29	185.57	9576.04	474.38	6242.83
31	185.95	10231.96	498.91	6752.41
33	186.25	11034.55	524.98	7272.38
35	186.58	12020.11	553.56	7842.47
37	186.94	13231.95	585.70	8483.38
39	187.37	14825.54	622.95	9226.37
41	187.89	17065.99	668.09	10126.65
43	188.53	19758.99	723.75	11236.81
45	189.25	23102.24	792.40	12553.74
47	190.01	26187.76	873.96	14080.81
49	190.77	<u>27180.12</u>	958.93	15685.41
51	191.39	26113.85	1032.96	17045.57
53	191.84	23965.73	1086.94	18037.53
55	192.10	21500.71	1118.68	18613.42
57	<u>192.18</u>	18990.37	<u>1129.73</u>	<u>18808.84</u>
59	192.13	16425.38	1122.27	18677.02
61	191.94	13922.40	1098.58	18251.41
63	191.63	11504.96	1061.19	17564.27
65	191.22	9310.96	1012.86	16676.28
67	190.75	7436.99	956.79	15646.09
69	190.21	5883.32	896.19	14507.45
71	189.63	4683.09	834.07	13315.01
73	189.07	3701.22	772.43	12191.10
75	188.40	2884.17	712.60	11014.39
77	187.75	2159.44	655.55	9876.53
79	187.13	1559.05	601.69	8802.35
81	186.55	1062.59	551.36	7798.61
83	186.02	730.10	505.0	6873.83
85	185.39	598.70	463.38	6013.65
87	184.82	566.41	427.01	5255.65

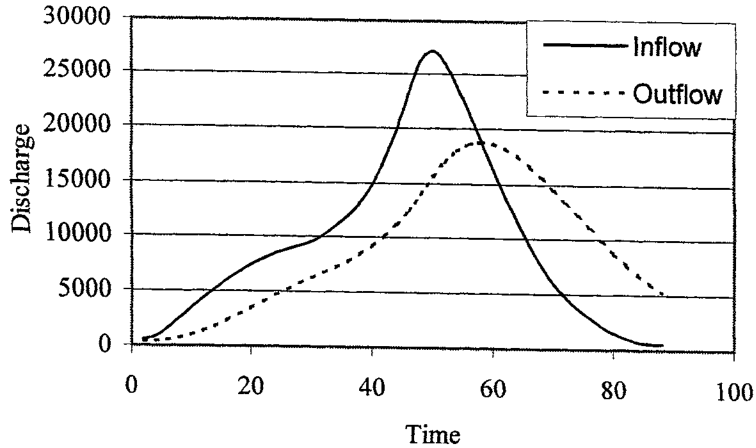


Fig. 10.13 Plot of Dharoi reservoir inflow and outflow for flood routing example.

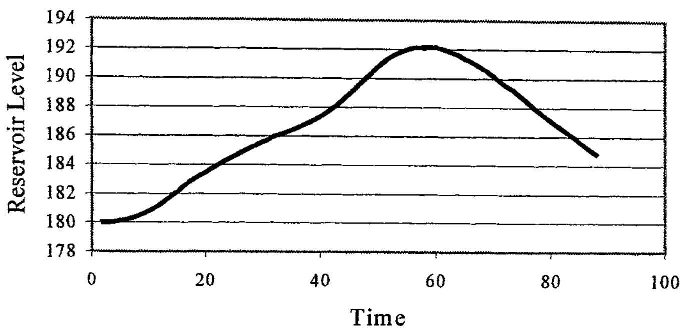


Fig. 10.14 Water level variation in the Dharoi reservoir for flood routing example.

10.13 FIXING TOP OF DAM

The top of a dam is fixed a little above the high flood level by providing extra space known as freeboard. Freeboard is the vertical distance between the maximum water level and the top of the dam. Freeboard is provided to ensure the safety of the dam against overtopping in the event of an adverse combination of hydro-meteorological variables, such as strong winds during a major storm which may push up the reservoir water level. The size of the freeboard depends on the design flood for freeboard, wind set-up, and the wave run-up that is expected depending on the length of the reservoir, prevailing winds, etc. When wind blows over the surface of a reservoir, water is piled up at the leeward end and is lowered at the windward end. The set-up is more pronounced in shallow reservoirs and can be calculated by (WMO 1994):

$$H_s = ku^2 \, l \, n \cos \theta / gd \tag{10.44}$$

where, H_s is the height of set-up above still pool level, u is the wind speed measured at an elevation of 10m above the still pool level, l is the fetch length (straight length of unobstructed water surface exposed to wind action), n is a dimensionless coefficient dependent upon the configuration and hydrology of lake, θ is the angle between wind direction and line along which fetch is measured, g is acceleration due to gravity, d is average water depth along the direction of wind, and k is a dimensionless shear-stress coefficient. For rectangular lakes of uniform depth, $n = 1$ and $k = 1.45*10^{-6}$ provided $u \leq 880d(g/l \cos \theta)$.

When wind blows across reservoirs, waves are generated which run up on the upstream face of dams. This run up is critical only when reservoir water level is near the top. Adequate freeboard is required to prevent overtopping of a dam by waves.

APPENDICES

10.A DEFINITIONS

a) Dead Storage Zone: This is the bottom most zone in a reservoir and the corresponding storage is also termed as inactive storage. Generally it is provided to cater for the sediment entering the reservoir, to provide minimum head for hydropower plants or to provide minimum pool for recreation facilities. Usually, all the outlets are located above this zone. The withdrawals from this zone, if any at all, are made only in extremely dry conditions. The entire reservoir storage which lies above the inactive storage is called live or active storage.

b) Buffer zone: This is the storage space on the top of the dead storage zone and the reservoir level is brought down to this zone under extreme drought situations. When the reservoir is in this zone, the release from the reservoirs caters only to the essential needs.

c) Conservation Zone: Water is stored in this zone to cater for various conservation requirements like irrigation, water supply and hydropower generation, etc. This zone, normally accounts for most of the storage space available in a conservation reservoir.

d) Flood Control Zone or Surcharge Zone: This zone is located on the top of conservation zone. The storage space of this zone is exclusively earmarked for absorbing or moderating floods impinging the reservoir. Depending on the flood volume and downstream release constraints, water is stored in this zone to attenuate a flood peak. After the flood peak has passed, this zone is emptied as soon as possible to prepare for subsequent flood events.

e) Spill zone: This storage space above the flood control zone corresponds to the flood rise during extreme floods and spilling. This space is occupied mostly during high flows and the releases are at or near maximum.

f) Full Reservoir Level (FRL): This is the highest level of the reservoir at which water is intended to be held for various conservation uses, including part or total of the flood storage without allowing any passage of water through the spillway.

g) Maximum Water Level (MWL): It is the highest level to which the reservoir water will rise while passing the design flood with the spillway facilities in full operation. This level refers to the top of the spill zone.

h) Active Storage: The storage space in a reservoir that is above the dead storage is termed as active storage. It is also called live storage.

i) Within-year Storage: Some reservoirs are operated to provide water over a short period of low flows only and therefore, these may fill-up and empty several times a year. The storage required in such a reservoir is known as within-year or seasonal storage (see Fig. 10.A1).

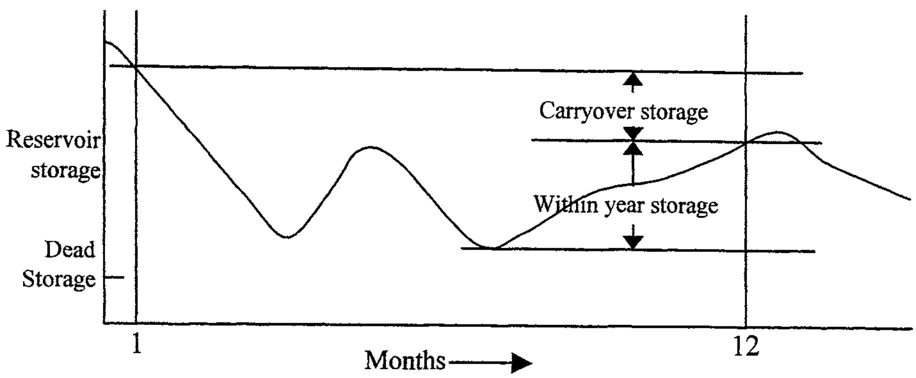


Fig. 10.A1 Carryover and within-year storages.

j) Carryover storage: When the water stored in a reservoir at the end of a year is carried over to the next year, this is called as carryover storage. This storage is estimated using annual data (ignoring seasonal fluctuations).

h) Release: Release or draft is the amount of controlled outflow from a reservoir during a given time interval to satisfy various demands. Release from the reservoir is also expressed as a ratio or percentage of mean inflow. This ratio is usually below 0.9; the low values are for regions where evaporation losses are higher and high values for regions where these losses are small.

i) Yield: For the reservoirs serving for conservation purposes, the amount of water released for these purposes is called the reservoir yield. For the reservoirs where the stored water is used to generate hydroelectric power, the yield is defined as the amount of power delivered during a time interval.

j) Firm Yield: Firm water yield from a reservoir is defined as the maximum quantity of water that can be guaranteed to be delivered with a 10% reliability (See Fig. 10.A2). The firm power yield of a reservoir can also be described in a similar manner. For example, if the yield of a reservoir for months of January to December is: 35, 42, 55, 67, 90, 75, 52, 33, 37, 40, 36, 41 units. Then, the firm yield will be the minimum of these values, i.e., 33 units.

k) Reliability: Reliability of a system (reservoir) is described by the probability α that the system is in the satisfactory state. A reservoir is in a satisfactory state if it meets all the demands. The reliability is given by:

$$\alpha = \text{Prob}[X_t \in S] \tag{10.A1}$$

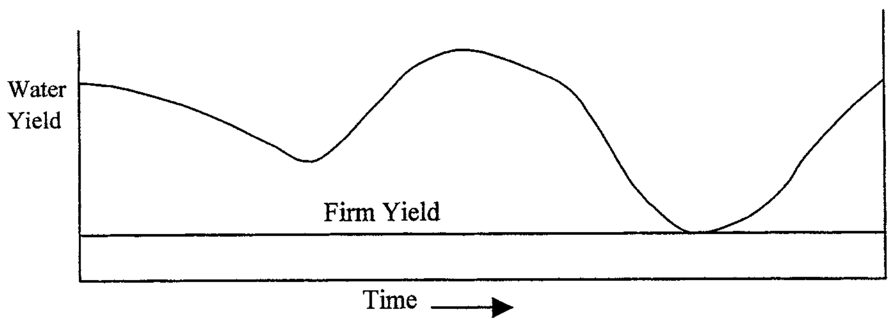


Fig. 10.A2 Variation of Yield and firm yield of a reservoir.

where X_t is the state of the system at time t and S is the domain of admissible states. Further, risk, which is the probability of failure, is $(1-\alpha)$. Thus, if a reservoir meets all the demands in 9 months of a year, its reliability will be 9/12 = 0.75 and the risk = 1.0 - 0.75 = 0.25.

Volume reliability (R_v) is the ratio of the volume of water supplied to that demanded:

$$R_v = \text{Volume of water supplied / Volume of water demanded} \tag{10.A2}$$

10.B FIXING LIVE STORAGE CAPACITY OF DHAROI RESERVOIR

The computations for preliminary fixing the live storage capacity of Dharoi reservoir are given in this appendix.

1. Irrigation (Direct Demand)		
a. October requirement		030.838 Mm^3
b. Fair weather (Nov. to June) requirement		145.557 Mm^3
	Total	176.395 Mm^3
2. Water supply demand for 9 months (October to June)		197.365 Mm^3
3. Fair weather (October to June) lake losses		088.197 Mm^3
	Total	461.957 Mm^3
4. Less post monsoon yield in the design year (1877)		013.939 Mm^3

Balance	448.018 Mm^3
5. Add carry over	196.995 Mm^3
Total	645.013 Mm^3
6. Add water supply reservation to ensure 99% reliability	137.662 Mm^3
7. Live storage	782.675 Mm^3
8. Add silt pocket of 131.988 Mm^3 at RL 175.87 m	131.988 Mm^3
9. Gross storage required	914.663 Mm^3
9a. Gross storage provided	907.878 Mm^3
10. Corresponding FRL	RL 189.60 m

The gross storage of the reservoir is computed to be 907.87 Mm^3 with FRL 622.0 m as detailed below:

Details of Project		Requirement (Mm^3)
A. Gross Utilisation		
a. Irrigation (396.606 Mm^2)		
i. Direct command requirement		218.335 Mm^3
b. Water Supply Demand		
i. Ahmedabad	7.8674 cumec	
Gandhinagar	0.5660 cumec	
Downstream riparian rights	0.8490 cumec	
Total	9.2824 cumec	197.365 Mm^3
ii. Water supply requirement of 9 months (October to June) less the availability from the run of river below Dharoi		
c. Lake Losses		144.323 Mm^3
d. Gross Utilization		560.023 Mm^3
e. Net Utilization (560.023 - 144.323)		415.70 Mm^3
f. Design Carry Over		196.995 Mm^3
g. Reservation for Water Supply		137.662 Mm^3
h. Design year 1877 having maximum carry over of 196.995 Mm^3		196.995 Mm^3
B. Storage Provision		
1. Irrigation (Direct Demand)		
a. October requirement		030.838 Mm^3
b. Fair weather (Nov. to June) requirement		145.557 Mm^3
Total		176.395 Mm^3
Say		197.365 Mm^3
2. Water supply demand for 9 months (October to June)		197.365 Mm^3
3. Fair weather (October to June) lake losses		088.197 Mm^3
Total		482.927 Mm^3
4. Less post monsoon yield in the design year (1877)		013.939 Mm^3
Balance		468.988 Mm^3
5. Add carry over.		196.995 Mm^3
Total		665.983 Mm^3

6. Add water supply reservation to ensure 99% reliability.	137.662 Mm^3
7. Live storage.	803.645 Mm^3
8. Add silt pocket of 131.988 Mm^3 R.L. 577.0	131.988 Mm^3
9. Gross storage required.	935.633 Mm^3
9.a. Gross storage provided.	907.878 Mm^3
10. F.R.L. required.	RL 622.20

The average utilisation of water for water supply and irrigation is worked out as under on the basis of the monthly reservoir working table of design year 1877.

Details of Project	Requirement in Mm^3
i. Annual water supply utilisation for October to June (Reliability 99%)	197.365 Mm^3
ii. Water supply reservoir (To ensure 99% reliability)	137.662 Mm^3
iii. Annual irrigation utilisation (Reliability 75%)	218.335 Mm^3
iv. Lake losses	144.323 Mm^3
v. Carry over	196.995 Mm^3

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