LECTURE NOTES – IV

« HYDROELECTRIC POWER PLANTS »

Prof. Dr. Atıl BULU

Istanbul Technical University
College of Civil Engineering
Civil Engineering Department
Hydraulics Division

CHAPTER 4

Potential Water Power

For any stretch of a watercourse, characterized by a difference in level of H meters, conveying a discharge of Q (m^3/sec), the *theoretical (potential) power*,

$$N_p = \gamma QH = 1000QH(kgm/\sec)$$
$$N_p = \frac{1000}{75}QH = 13.3QH(HP)$$
$$N_p = 13.3 \times 0.736QH = 9.8QH(kW)$$

If the rate of flow changes along a stretch, the mean value of the discharges pertaining to the two terminal sections of the stretch is to be substituted in the equation, $Q = (Q_1 + Q_2)/2$,

The theoretical power resources of any river or river system are given by the total of the values computed for the individual stretches,

$$N_p = \sum \frac{1000QH}{75} \times 0.736 = 9.8 \sum QH(kW)$$

Potential water power resources can be characterized by different values according to the discharge taken as basis of computation. The conventional discharges are,

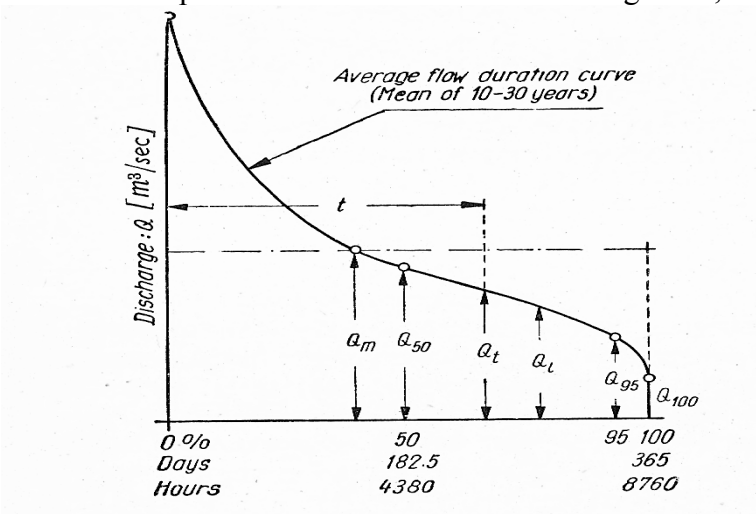


Figure. Discharges used for characterising potential water power resources

1. **Minimum potential power**, or theoretical capacity of 100%, is the term for the value computed from the minimum flow observed. N_{p100}
2. **Small potential power**. The theoretical capacity of 95% can be derived from the discharge of 95% duration as indicated by the average flow duration curve. N_{p95}.
3. **Median** or **average potential power**. The theoretical capacity of 50% can be computed from the discharge of 50% duration as represented by the average flow duration curve. N_{p50}.
4. **Mean potential power**. The value of theoretical mean capacity can be ascertained by taking into account the average of mean flow. The average of mean flow is understood as the arithmetic mean of annual mean discharges for a period of 10 to 30 years. The annual mean discharge is the value that equalizes the area of the annual flow duration curve.

Economic significance of potential water resources of a site.

This is influenced by a great number of factors than hydraulic, such as geographical, geological and topographical conditions, energy demand, etc. Ignoring these and comparing relative values of power potential as reflected by hydraulic conditions only, the following four aspects are to be taken into consideration:

a) **The absolute quantity of theoretical water power resources,**

b) **The relative share of discharge in the power.**
Among the hydraulic possibilities representing equal magnitudes, the more advantageous are those where the power in question originates from a smaller flow and a higher head. It is advantageous of highland developments over power stations situated in hilly regions or lowland areas.

c) **The relative annual fluctuation of available potential power.**
This can be characterized by the ratio of the values N_{p50} to the values N_{p95} (or N_{p100}).

$$\alpha = \frac{N_{p50}}{N_{p95}} \quad \text{or,} \quad \alpha_1 = \frac{N_{p50}}{N_{p100}}$$

A smaller ratio reflects a more favorable hydraulic possibility.

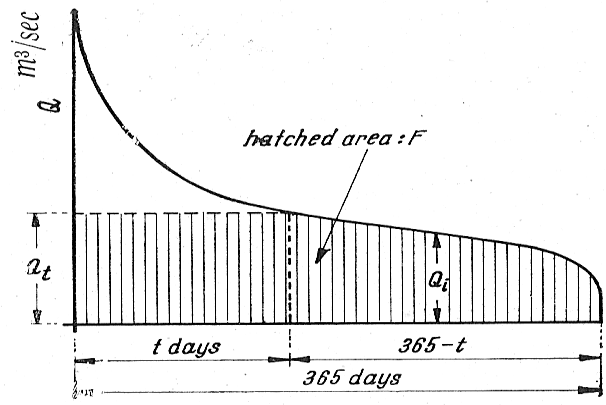
d) **The over year or multi-annual variation of potential power.**

This can be characterized either by a simple diagram showing the annual potential power against time, or by a summation curve of annual values.

Power resources can be characterized even by *annual values of potential energy* in a river, by the quantities of work,

$$E_{100}\ ,\ E_{95},\ E_{50} \text{ and } E_m$$

All expressed in kilowatt-hours. These values can be computed as areas of the lower parts of the *potential power-duration curves*. If the head is assumed to be constant, independent of discharge, the computation can be based on the discharge-duration curve. Using the curve,



Figure

$$N_p = 9.8HQ_t = aQ_t(kW)$$

$$E = 24aQ_t t + 24\sum_{t}^{365} aQ_i$$

$$E = 24a\left(Q_t t + \sum_{t}^{365} Q_i\right) = 24aF(kWh)$$

Where,

t = the duration considered in days,
Q_t = Selected discharge,
Q_i = Daily mean of actual discharge at any time,
F = Area pertaining to Q_t (shaded area).

The upper limit of potential energy inherent in the river section is obtained by,

$$E_{mx} = (24 \times 365)N_m = 8760N_m(kWh)$$

Where N_m is the annual mean power.

The overall coefficient is about 0.75 or 0.80. The equation recommended is,

$$N_{mnet} = (7.4 - 8.0)\sum Q_m H$$

Q_m = the arithmetic mean discharge. Net river energy potential,

$$E_{\max_{net}} = 8760\sum N_{m_{net}}$$

For characterizing the *gross potential power of a river basin*, the following data should be used,

a) Total annual discharge volume V (m^3),
b) Medium height of the watershed area H lying upstream above sea level (m),
c) Area A of this basin (km^2).

Since 1 m^3 water weighs 1 ton, the product VH yields the annual gross power pertaining to the selected site in meter-tons,

$$E(kWh) = \frac{VH}{367}$$

$$e(kWh/km^2) = \frac{VH}{367A}$$

If the e values are determined along the river basin at several stream sites, then the lines connecting the points of equal e, *isopotential lines* can be drawn.

Flow-Duration Curve

If the flows for any unit time are arranged in descending order of time (without regard to chronological sequence), the percentage of time for which any magnitude is equaled or exceeded may be computed. The resulting array is called a *flow-duration curve***.**

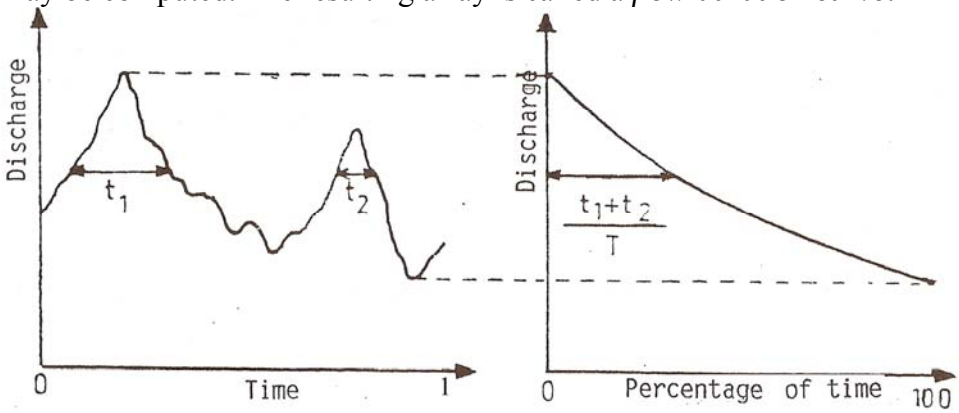


Figure. Determination of flow-duration curve (Bayazıt, 2001).

Such curves are useful in determining the relative variability of flow between two points in a river basin or between two basins. For example, if a stream is highly regulated, the curve will approach a horizontal line. The dependable flow is that corresponding to 100 percent of time. The relative variability of two flow records may be compared by converting the discharge scale in terms of a ratio to the mean. Any sub area under the curve represents the volume of annual runoff.

Flow-duration curves have been used to approximate the amount of storage needed to increase the dependable flow. For example, the horizontal line AB in below Figure may represent a new dependable flow, and the required storage needed to obtain this flow is indicated by area ABC.

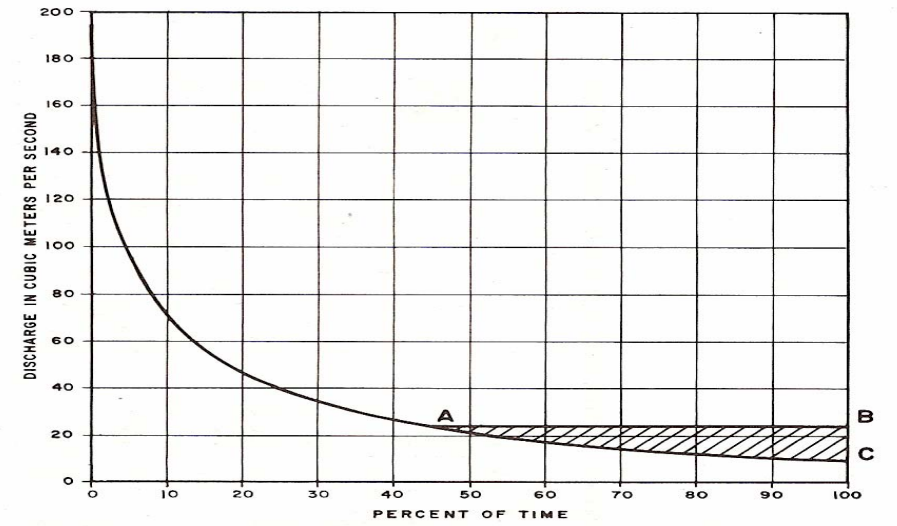


Figure . Duration curve of monthly discharges.

Power production values may be approximated from the duration curve by converting the discharge scale to kilowatts by multiplying by a selected head, efficiency and conversion factors. If the time scale is converted to hours in a year, a unit of are represents kilowatt-hours.

The flow-duration curve is particularly useful in combination with a sediment rating curve (river discharge versus the transported sediment load usually expressed in tons per day), to compute total sediment load to be expected in an average year.

Flow Mass Curve

Total flow volume from a certain initial time t = 0 up to time t can be computed as,

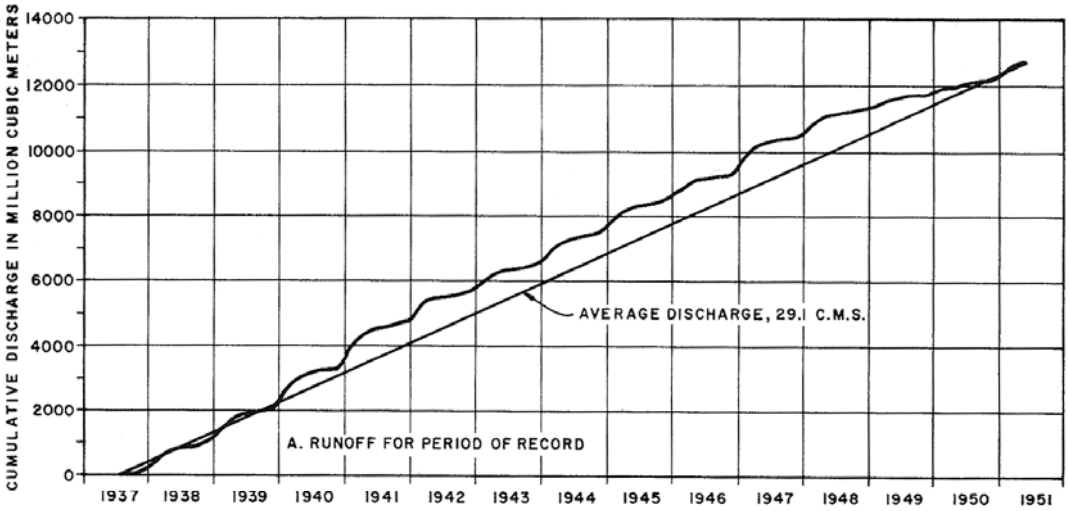
$$H = \int_0^t Q dt$$

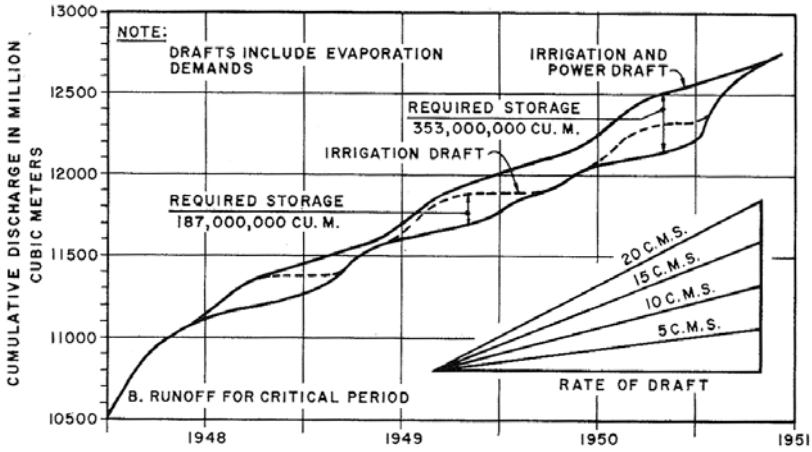
In practice, the total volume is computed as,

$$H = \sum Q_i t_i$$

Q_i = the average discharge in time interval (month, year) Δt_i.

Flow mass curve is a plot of the cumulative runoff from the hydrograph against time. The time scale is the same as for the hydrograph and may be in days, months or years. The volume ordinate may be in m^3-days, m^3-months, m^3-years, etc. The slope of the mass curve is the derivative of the volume with respect to time or the rate of discharge.





The mass curve usually has a wavy configuration in which the steeper segments represent high flow periods and flatter segments represent low flows. Uniform rates of withdrawal (draft) may be represented as tangent lines drawn from high points to intersect the curve at the next wave. The vertical distance between the draft line and the basic curve represents the cumulative difference between regulated outflow and natural inflow, or the *required storage*. If the draft line does not intersect the mass curve at the end of a year, it means that the reservoir does not refill with that rate of draft and regulation at the proposed draft rate will extend over two years or more. A typical mass curve is shown in the above Figure.

In estimating storage requirements from the mass curve, it is not necessary to assume a constant rate of regulated flow. For example, if the draft rate to meet a demand for irrigation, water supply, or power varies from month to month, the draft line may be a curved or irregular line and the maximum draft may not occur at the low point in the mass curve.

An allowance for evaporation should be applied to the mass curve analysis. If the water area does not change significantly during the annual cycle of use, an average correction for each calendar month can be subtracted from the inflow or added to the draft rates.

The ordinates of the flow mass curve increase continuously in time. The sum of the differences between the inflow and the yield (average flow) are drawn;

$$H_0 = \sum (Q_i - Q_{ave}) \Delta t_i$$

Reservoir capacity is then vertical distance between the highest and lowest points of the curve.

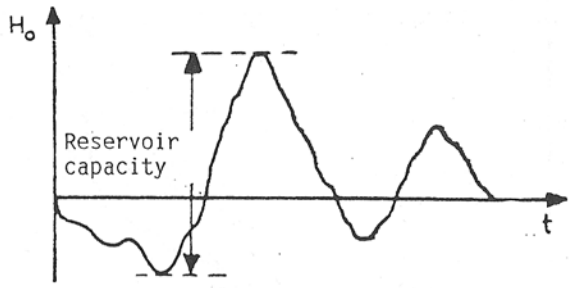


Figure . Flow mass curve derived using the differences of the discharges from the yield.

Storage-Draft Curve

The results of a mass-curve analysis can be plotted as a *storage-draft curve***.** His curve gives the storage needed to sustain various draft rates. Examples of storage-draft curves are shown in the below Figures. Both irrigation requirements and combined irrigation and power requirements are illustrated. These curves were computed from the mass curve.

If storage unlimited, the storage-draft curve will approach the available mean flow as asymptote. It is rarely possible to develop mean annual flow of a river basin. For most projects, some spillage will occur in years of runoff. To impound all flood flows will require an extensively large reservoir. Such a reservoir may not fill in many years, and probably could not be justified economically. The selected rate of regulated flow to be developed will depend on;

1. The demand of water users,
2. The available runoff,
3. The physical limits of the storage capacity,
4. The overall economies of the project.

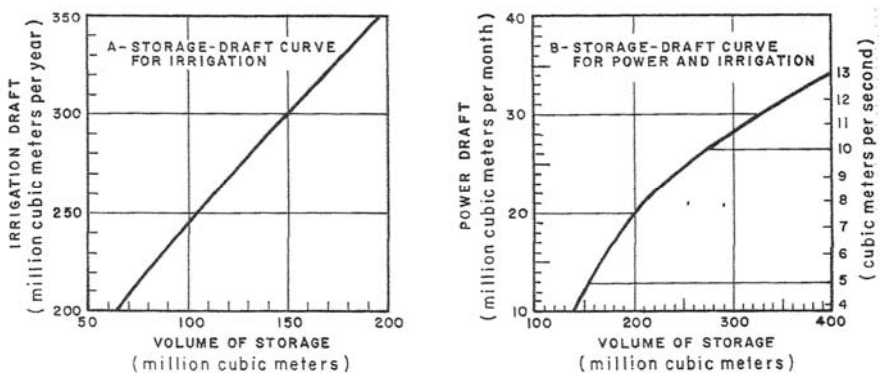


Figure . Storage-draft curves for multipurpose uses.

Selection of Design Flow

The hydrologic analyses, combined with economic analyses of costs and benefits for different heights of dam and reservoir capacity will lead to the selection of the reservoir capacity and the corresponding dependable flow that can be justified. The selected design flow may not necessarily be available 100 percent of the time. The propose water use may permit deficiencies at intervals, for example, a 15 percent shortage once in 10 years. Irrigation water supplies may permit greater deficiencies than those for urban and industrial use. Hydroelectric power plants, connected to large systems, may tolerate substantial water supply deficiencies.

Final Storage Selection

a) Evaporation Losses

Detailed evaluation of evaporation losses should be postponed until final operation and routing studies, when the actual variation in water area can be considered as well as the seasonal variation in evaporation.

Basic data on water surface evaporation may be obtained from records of pan evaporation. Such records overestimate lake evaporation and must be reduced by a *pan coefficient* which varies from 0.60 to 0.80 depending on the climate. The collection of evaporation records at a project site should be initiated in the planning stage. Evaporation corrections should be made on a monthly basis using actual past precipitation records at the project site if possible.

b) Power

Selection of an average flow alone will not permit determination of the benefits from a water resources development project without more detailed studies. Such studies require routing through the reservoir the entire record of flow (corrected for evaporation losses), on a month by month basis, using assumed patterns of use, outlet capacities and, in the case of power, turbine and generator capacities and efficiencies. The reservoir would normally be considered to be full at the start of the operation study, or at least full to normal pool.

For power benefits, the energy output will vary in accordance with the inflow, outflow, and change in storage and corresponding head, tailwater elevation, turbine capacity and plant efficiency. If the plant is a part of a system, the output may be subject to varying demands of the system load curve and whether the plant is to be used as a base load plant or a peaking plant. The routing study will indicate the necessary modifications to the head, storage, and even height of the dam to obtain maximum benefit.

c) Irrigation

Operation studies for irrigation use should be made using seasonal crop demands and selected outlet capacities. Short-term demands may indicate that the storage needed war greater than that required for uniform regulated flow. The proposed annual water use may be greater than that available 100 percent of the time, with the understanding that deficiencies can be tolerated in some years.

d) Water Supply

Operation studies for projects providing urban water supplies will be similar to those for irrigation projects in that there may be variations in the seasonal demand, especially where more than one source is available, or where there can be transfers to other regulation reservoirs. However, the degree of dependability of flow must be higher for urban water supply than for irrigation projects.

e) Flood Control

The storage allocated for flood control in single purpose or multipurpose projects is usually based on a definite design flood the control of which is needed for downstream protection. The required storage capacity is based on routing of the design flood inflow coincident with releases not to exceed downstream channel capacities.

Total Storage Requirement

The usable storage needed for single purpose projects can be readily determined as described in Sections (a) to (e). The total usable storage needed for multipurpose for multipurpose projects require more complex routing studies and numerous trials to obtain the most economic allocations.

In addition to the variable requirement for storage for downstream uses, the total storage may be increased for the following reasons:

- Minimum head on power installations.
- Allowance for the storage of sediments without loss of usable storage.
- Minimum area for recreation use, including seasonal requirements.

EXAMPLE : Monthly flow volumes feeding a reservoir are given in the table. Determine the storage capacity required to supply the mean annual flow volume yield.

Solution: Cumulative volumes are calculated and given in the table.

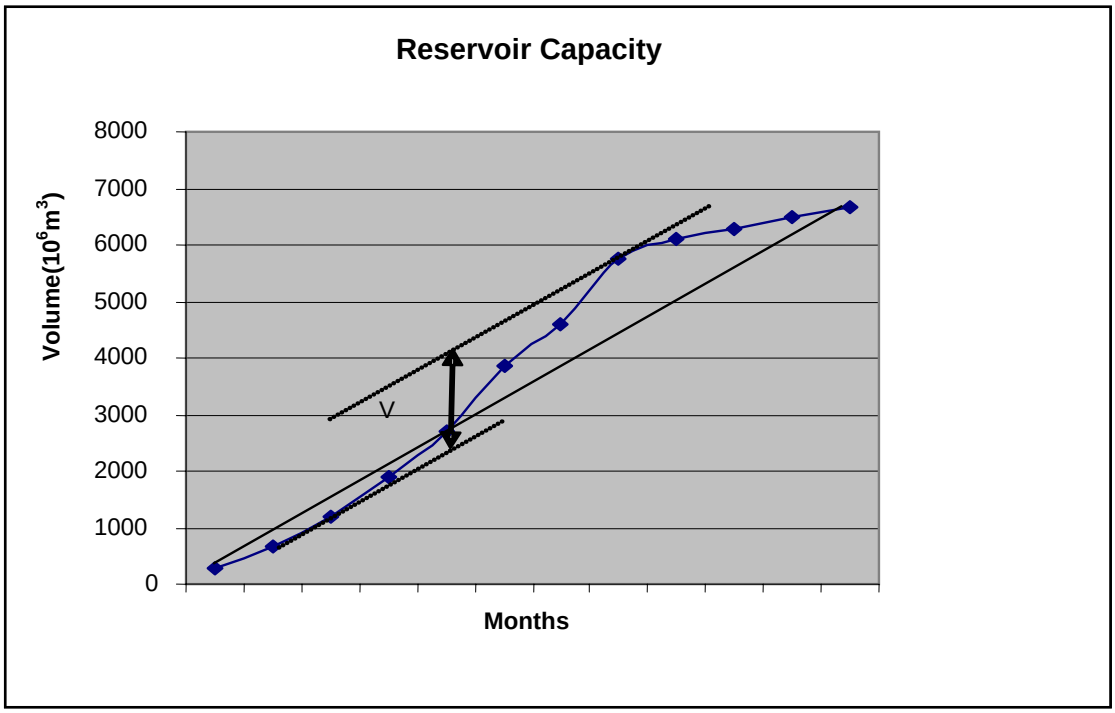
Month	Volume ($10^6 m^3$)	Cumulative Volume ($10^6 m^3$)
1	296	296
2	386	682
3	504	1186
4	714	1900
5	810	2710
6	1154	3864
7	746	4610
8	1158	5768
9	348	6116
10	150	6266
11	223	6489
12	182	6671

Total volume of flow feeding the reservoir is 6671×10^6 m^3. Annual mean discharge can be calculated as,

$$Q = \frac{6671\times10^6}{365\times86400} \cong 212\, m^3/s$$

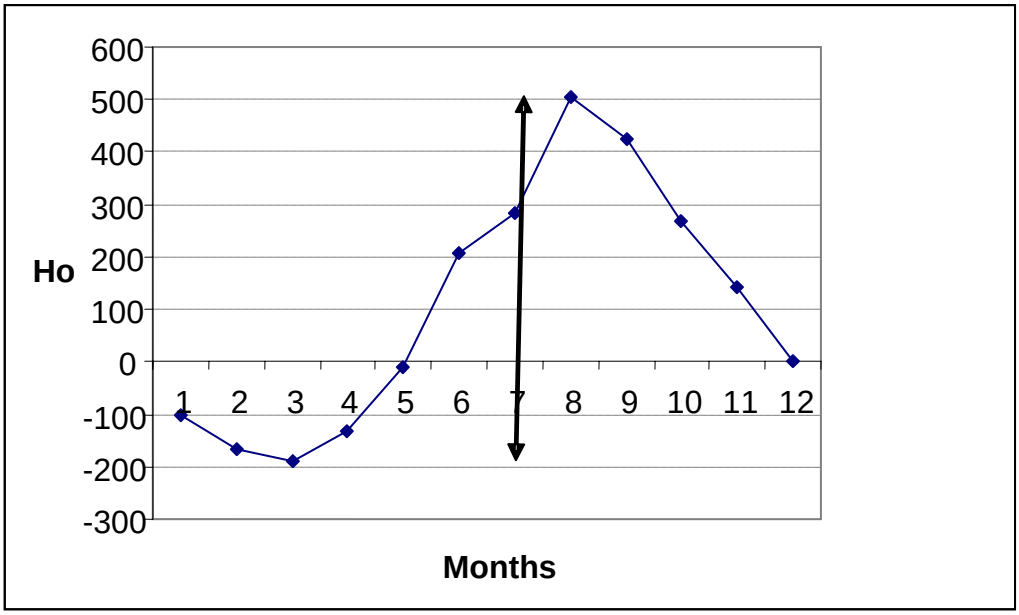
The reservoir storage capacity required to obtain 212 m^3/s yield throughout the year is found by drawing tangents parallel to the average draft line from peak points. The vertical distance is 1800×10^6 m^3 is the required capacity of the reservoir.

The reservoir capacity to supply the annual mean discharge can be found out by using the sum of differences method as in table,



Month	Volume (10^6m^3)	Flow (m^3/s)	H_0 (m^3/s)	ΣH_0 (m^3/s)
1	296	111	-101	-101
2	386	149	-63	-165
3	504	188	-24	-188

4	714	267	55	-134
5	810	335	123	-11
6	1154	431	219	208
7	746	288	76	284
8	1158	432	220	504
9	348	134	-78	426
10	150	56	-156	270
11	223	83	-129	142
12	182	70	-142	0



$$H_0 = \sum (Q_i - Q_{ave}) \Delta t_i$$

Reservoir Capacity = 504 – (-188) = 693 m^3/s

Volume = $692 \times 31 \times 86400 = 1853 \times 10^6 m^3$

RESERVOIRS

A reservoir is a manmade lake or structure used to store water. A dam reservoir has an uncontrolled inflow but a largely controlled outflow. The water available for storage is totally a function of the natural stream streamflow that empties into it.

Reservoir Capacity

Reservoir capacity is the volume of water that can be stored in the particular reservoir. It is the *normal maximum pool* level behind a dam. This can be calculated by using a topographic map of the region. First, the area inside different elevation contours is measured, and then a curve of area versus elevation can be constructed.

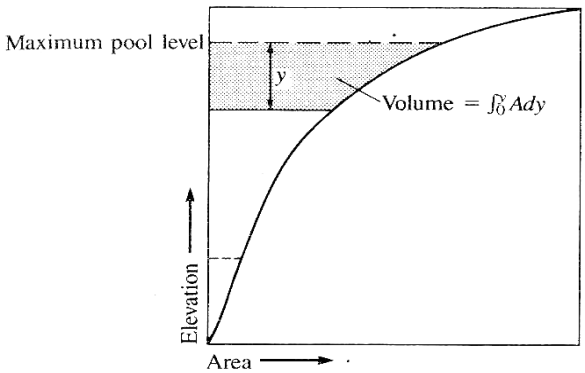


Figure . Area versus elevation for a reservoir

At any given elevation, the increment of storage in the reservoir at that elevation will be Ady, where dy is a differential depth. Then the total storage below the maximum level to any will be given by,

$$V = \int_0^y Ady$$

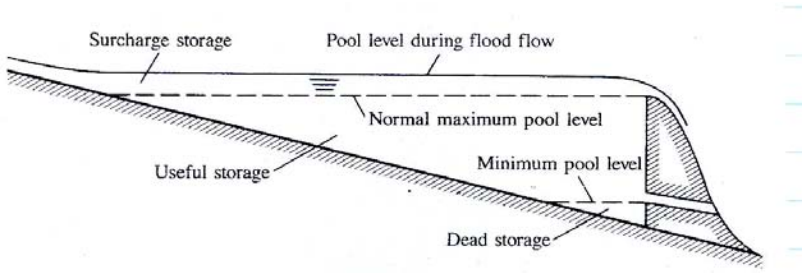


Figure. Storage relations for a reservoir with an uncontrolled spillway

Sedimentation in Reservoirs

All streams carry sediments that originate from erosion processes in the basins that feed the streams. After a dam is constructed across the steam and a reservoir is produced, the velocity in the reservoir will be negligible so that virtually all the sediment coming into the reservoir will settle down and be trapped. Therefore, the reservoir should be designed with enough volume to

hold the sediment and still operate as a water storage reservoir over the project's design life. For large projects, the design life is often considered 100 years.

Sediment carried in a stream is classified as either *bed load* or *suspended load*. The bed load consists of the coarsest fractions of the sediment (sands and gravels), and it rolls, slides, and bounces along the bottom of the stream. The finer sediments are suspended by the turbulence of the stream. When the sediment enters the lower velocity zone of the reservoir, the coarser sediments will be deposited first, and it is in this region that a delta will be formed. The finer sediments will be deposited beyond the delta at the bottom of the reservoir.

The total sediment outflow from a watershed or drainage basin measured in a specified period is the *sediment yield*. The yield is expressed in terms of tons per square kilometer per year. The engineer designing a reservoir must estimate the average sediment yield for the basin supplying the reservoir to determine at what rate the reservoir will fill with sediment.

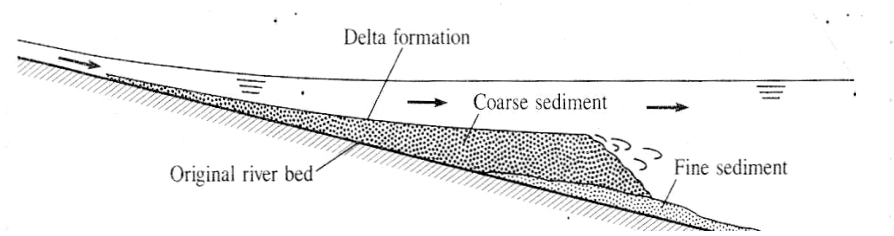


Figure. Deposition of sediment in a reservoir

For a given reservoir volume, V, the ratio of the reservoir volume decrease due to the deposited sediment can be estimated by this empirical equation,

$$R = 23\times10^{-6} G^{0.95}\left(\frac{A}{V}\right)^{0.8}$$

Where,

G = Sediment yield of the basin (kN/km^2/year)
A = Drainage basin area (m^2)
V = Reservoir volume (m^3)

Multiplying R ratio with the design life of the reservoir, T, will yield the percentage of the dead volume in the reservoir. The dead volume can be estimated over the period of design life by,

$$V_{dead} = R \times T \times V_{reservoir}$$

EXAMPLE 4.3: The total volume of a reservoir is V = 230×10^6 m^3 with a drainage basin of A = 1200 km^2. The design life of the project is T = 100 year and the density (specific mass) of the deposit is ρ = 2.65 ton/m^3. Calculate the dead volume of the reservoir.

Solution:

The sediment of the river for a river,

$$G = 1421A^{-0.229}$$
$$G = 1421 \times 1200^{-0.229}$$
$$G = 280\left(m^3 / km^2 / year\right)\left(volume\right)$$

The ratio of the reservoir volume decrease every year,

$$R = 0.000023 \times G^{0.95}\left(\frac{A}{V}\right)^{0.8}$$
$$R = 0.000023 \times 7279^{0.95} \times \left(\frac{1200 \times 10^6}{230 \times 10^6}\right)^{0.8}$$
$$R = 0.4\left(\%\right)$$

Reservoir volume decrease due to the sediment deposit every year,

$$V_{dead} = 0.004 \times 230 \times 10^6 = 0.92 \times 10^6 m^3$$

For the 100 year of design period,

$$V_{dead_{100}} = 100 \times 0.92 \times 10^6 = 92 \times 10^6 m^3$$

Useful storage is,

$$V_{useful} = \left(230 - 92\right) \times 10^6 = 138 \times 10^6 m^3$$

Wind-Generated Waves, Setup, and Freeboard

Whenever wind blow over an open stretch of water, waves develop, and the mean level of the water surface may change. The latter phenomenon, called *setup* or *wind tide*, is significant only in relatively shallow reservoirs. When a dam is designed, the crest of the dam must be made higher than the maximum pool level in the reservoir to prevent overtopping of the dam as the wind-generated waves strike the face of it. The additional height given to the crest of the dam to take care of wave action, setup, and possibly settlement of the dam (if it is earthfill) is called *freeboard*.

Setup

Consider the basin of water shown in the figure. The solid line depicting the water surface is the case when no wind is blowing; the water surface is horizontal. When the wind is blowing, a shear stress acts on the water surface, and because of this, the surface will tilt, as shown by the broken line in the below Figure.

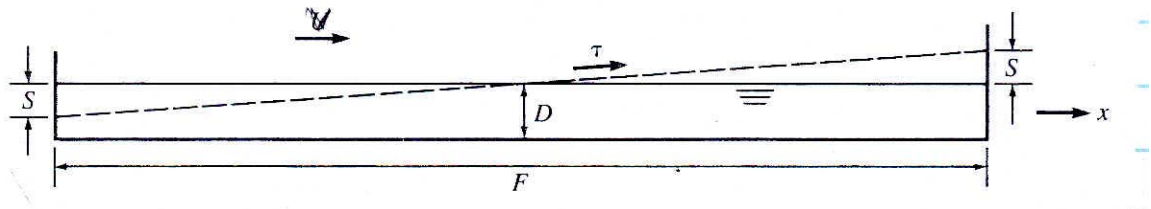


Figure. Definition sketch for setup

The amount of setup S is,

$$S = \frac{V^2 F}{KD}$$

Where,

D = Average reservoir depth (m),
V = the wind speed measured at a height of 10 m from the surface (km/h)
F = the wind fetch (km)
K = A constant ≈ 62000
S = Setup (m)

EXAMPLE: A reservoir is oval shaped with a length of 20 km and a width of 10 km. If the wind blows in a direction lengthwise to the reservoir with a velocity of 130 km/h, what will be the setup of the average water depth of the reservoir is 10 m?

Solution: The setup will be,

$$S = \frac{V^2 F}{KD}$$

$$= \frac{130^2 \times 20}{62000 \times 10} = 0.55m$$

Height of Wind Waves and the Run-Up

Allowances for wave height and the run-up of wind-generated waves are the most significant components of freeboard. The *run-up* of the waves on the upstream dam face, i.e. the maximum vertical height attained by a wave running up a dam face, is equal to H (wave height) for a typical vertical face in deep water, but can attain values over 2H for a smooth slope 1 in 2.

A *wave height*, H (m), (crest to trough) can be estimated by,

$$H = 0.34\sqrt{F} + 0.76 - 0.26\sqrt[4]{F}$$

Where,

F = the fetch length (km),
H = Wave height (m)

For large values of fetch (F>20 km), the last two terms may be neglected. With the provision for the wind speed, the equation takes the form of,

$$H = 0.032\sqrt{VF} + 0.76 - 0.24\sqrt[4]{F}$$

Where,

V = Wind velocity (km/hour)

The freeboard will be equal to set-up plus run up allowance for settlement of the embankment plus and amount of safety (usually 0.50m).

EXAMPLE: Calculate the wind set-up and wave height for a reservoir with 8 km fetch length. The average reservoir depth is 15 m. The wind velocity is V = 100 km/h. If the upstream of the dam is vertical, what will be the minimum freeboard to be given?

Solution:

The wind set-up is,

$$S = \frac{V^2 F}{KD}$$
$$= \frac{100^2 \times 8}{62000 \times 15} = 0.09m$$

The wave height,

$$H = 0.032\sqrt{VF} + 0.76 - 0.24\sqrt[4]{F}$$
$$H = 0.032\sqrt{100 \times 8} + 0.76 - 0.24\sqrt[4]{8}$$
$$H = 1.19m$$

Since the upstream side of the dam is vertical, the run-up height will be taken as the height of the wave. The freeboard,

$$H_{freeboard} = 0.09 + 1.19 + 1.19 + 0.5$$
$$H = 2.97m \cong 3.00m$$

0.5 m is the safety height.