

Reservoirs and Planning for Dam Reservoirs

18.1. Definition and Types

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, generally called a *dam reservoir* or an *impounding reservoir* or a *river reservoir*.

The quality of water stored in such a reservoir is not much different from that of a natural lake. The water so stored in a given reservoir during rainy season can be easily used almost throughout the year, till the time of arrival of the next rainy season, to refill the emptying reservoir again.

Depending upon the purpose served by a given reservoir, the reservoirs may be broadly divided into the following three types.

- (1) *Storage or Conservation reservoirs* ;
- (2) *Flood Control reservoirs* ; and
- (3) *Multipurpose reservoirs*.

The fourth type of a reservoir is a simple storage tank constructed within a city water supply system, and is called a *Distribution reservoir* ; and such a reservoir is evidently not a river reservoir, but is a simple storage tank.

18.1.1. Storage or Conservation Reservoirs. A city water supply, irrigation water supply, or a hydroelectric project drawing water directly from a river or a stream may fail to satisfy the consumers demands during extremely low flows; while during high flows, it may become difficult to carry out their operations due to devastating floods. *A storage or a conservation reservoir can retain such excess supplies during periods of peak flows, and can release them gradually during low flows as and when the need arises.*

Incidentally, in addition to conserving water for later use, the storage of flood waters may also reduce flood damage below the reservoir. Hence, a reservoir can be used for controlling floods either solely or in addition to other purposes. In the former case, it is known as a 'Flood-Control-Reservoir' or a 'Single-Purpose Flood Control Reservoir'; and in the latter case, it is called a 'Multipurpose Reservoir'.

18.1.2. Flood Control Reservoirs. A flood control reservoir, generally called a flood-mitigation reservoir, stores a portion of the flood flows in such a way as to minimise the flood peaks at the areas to be protected downstream. To accomplish this, the entire inflow entering the reservoir is discharged till the outflow reaches the safe capacity of the channel downstream. The inflow in excess of this rate is stored in the

reservoir, which is then gradually released, so as to recover the storage capacity for the next flood.

The flood peaks at the downstream of the reservoir are thus reduced by an amount AB , as shown in Fig. 18.1. A flood control reservoir differs from a conservation reservoir only in its need for a large sluiceway capacity to permit rapid drawdown before or after a flood.

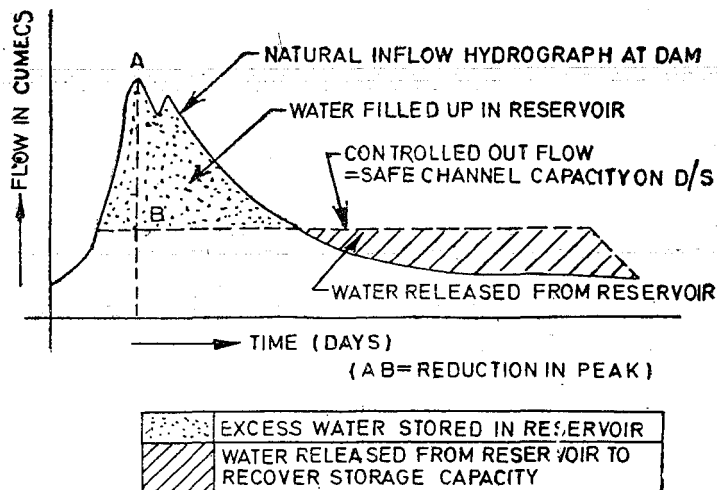


Fig. 18.1.

Types of flood control reservoirs. There are two basic types of flood-mitigation reservoirs; i.e.

- (i) *Storage reservoirs or Detention basins*; and
- (ii) *Retarding basins or Retarding reservoirs.*

A reservoir having gates and valves installation at its spillway and at its sluice outlets is known as a storage reservoir; while on the other hand, a reservoir with uncontrolled and ungated outlets is known as a **retarding basin**.

Functioning and Advantages of a Retarding Basin

A retarding basin is usually provided with an uncontrolled spillway and an uncontrolled orifice type sluiceways. The automatic regulation of outflow, depending upon the availability of water, takes place from such a reservoir. The maximum discharging capacity of such a reservoir should be equal to the maximum safe carrying capacity of the channel downstream. As floods occur, the reservoir gets filled, and discharges through sluiceways. As the reservoir elevation increases, the outflow discharge increases. The water level goes on rising until the flood has subsided, and the inflow becomes equal to or less than the outflow. After this, the water gets automatically withdrawn from the reservoir until the stored water is completely discharged. The advantages of a retarding basin over a gate controlled detention basin are:

- (i) Cost of the gate installation is saved.
- (ii) There are no gates and hence, the possibility of human error and negligence in their operation is eliminated.

- (iii) Since such a reservoir is not always filled, much of the land below the maximum reservoir level will be submerged only temporarily and occasionally, and can be successfully used for agriculture, although no permanent habitation can be allowed on this land.

Functioning and Advantages of a Storage Reservoir

A storage reservoir with gated spillway and gated sluiceways, provides more flexibility of operation, and thus gives us better control and increased usefulness of the reservoir. Storage reservoirs are, therefore, preferred on large rivers, which require better control; while retarding basins are preferred on small rivers. In storage reservoirs, the flood crest downstream, can be better controlled and regulated properly, so as not to cause their coincidence. This is the biggest advantage of such a reservoir and outweighs its disadvantages of being costly and involving risk of human error in installation and operation of gates.

18.1.3. Multipurpose Reservoirs. A reservoir planned and constructed to serve not only one purpose but various purposes together is called a multipurpose reservoir. Reservoir, designed for one purpose, incidentally serving other purposes, shall not be called a multipurpose reservoir, but will be called so, only if designed to serve those purposes also in addition to its main purpose. Hence, a reservoir designed to protect the downstream areas from floods and also to conserve water for water supply, irrigation, industrial needs, hydroelectric purposes, etc. shall be called a multipurpose reservoir. Bhakra dam and Nagarjun Sagar dam are the important multipurpose projects of India.

18.1.4. Distribution Reservoirs. A distribution reservoir is a small storage reservoir constructed within a city water supply system. Such a reservoir can be filled by pumping water at a certain rate and can be used to supply water even at rates higher than the inflow rate during periods of maximum demands (called critical periods of demand). Such reservoirs are, therefore, helpful in permitting the pumps or the water treatment plants to work at a uniform rate, and they store water during the hours of no demand or less demand, and supply water from their 'storage' during the critical periods of maximum demand.

In this chapter, we shall however, confine ourselves to the river reservoirs only.

18.2. Capacity-Elevation and Area-Elevation Curves of a Reservoir Site

Whatever be the size or use of a reservoir, the main function of a reservoir is to store water and thus to stabilize the flow of water. Therefore, the most important physical characteristic of a reservoir is nothing but its *storage capacity*. The capacity of reservoirs on dam sites, is determined from the contour maps of the area. A topographic survey of the dam site is carried out, and a contour map such as shown in Fig. 18.2 is drawn. The area enclosed within each contour can be measured with a planimeter.

In fact, the general practice adopted for capacity computations is to actually survey the site contours only at vertical distances of 5 m or so. The areas of the intervening contours at smaller intervals of say 0.5 m or so, are then interpolated by taking the square root of the surveyed contours, and to assume that the square root of the interpolated ones, vary in exact proportion to their vertical distance apart. For example, suppose the area of the reservoir at 200 m contour is A_1 hectares, and that at 205 m contour is A_2 hectares; then

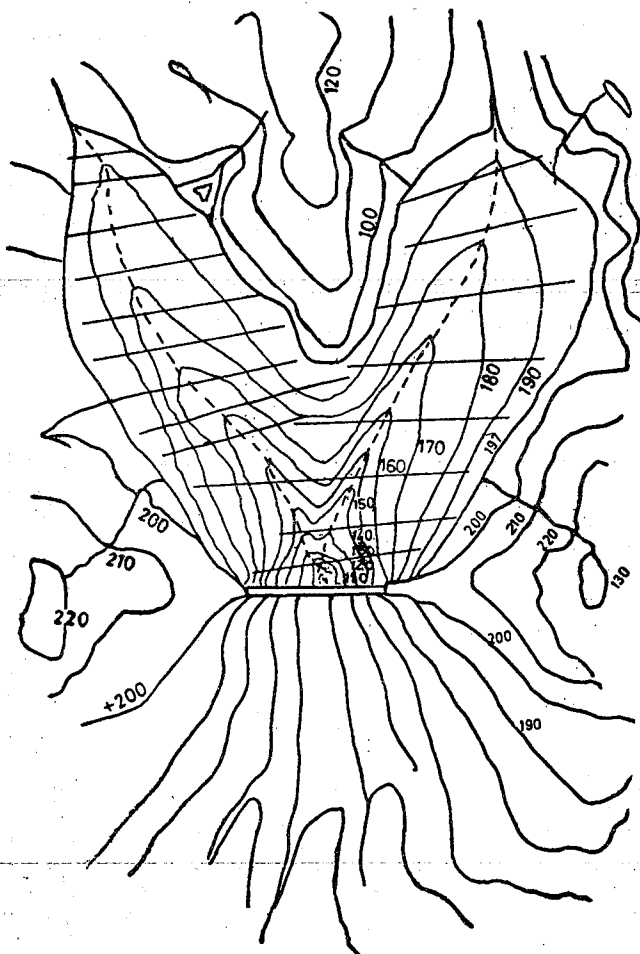


Fig. 18.2. A typical contour map of a dam site.

The area at 200.5 m contour is

$$\begin{aligned}
 &= \left[\sqrt{A_1} + \frac{200.5 - 200}{205 - 200} (\sqrt{A_2} - \sqrt{A_1}) \right]^2 \\
 &= \left[\sqrt{A_1} + \frac{0.5}{5.0} (\sqrt{A_2} - \sqrt{A_1}) \right]^2 \quad \dots [18.1 (a)]
 \end{aligned}$$

Similarly, the area at 201.0 m contour

$$\begin{aligned}
 &= \left[\sqrt{A_1} + \frac{201.0 - 200}{205 - 200} (\sqrt{A_2} - \sqrt{A_1}) \right]^2 \\
 &= \left[\sqrt{A_1} + \frac{1.0}{5.0} (\sqrt{A_2} - \sqrt{A_1}) \right]^2 \quad \dots [18.1 (b)]
 \end{aligned}$$

Similarly, the area at 204.5 m contour

$$\begin{aligned}
 &= \left[\sqrt{A_1} + \frac{204.5 - 200}{205 - 200} (\sqrt{A_2} - \sqrt{A_1}) \right]^2 \\
 &= \left[\sqrt{A_1} + \frac{4.5}{5.0} (\sqrt{A_2} - \sqrt{A_1}) \right]^2 \quad \dots [18.1 (c)]
 \end{aligned}$$

In this way, the areas can be computed at sufficiently low contour intervals (0.5 m), which can be used to determine the incremental volumes (ΔS) stored between two successive contours, by using the simple average method, *i.e.* by multiplying the average of the two areas at the two elevations, by the elevation difference (Δh). The summation of these incremental volumes below any elevation, is the storage volume below that level.

Instead of using the simple average formula, *i.e.*

$$\Delta S = \frac{a_1 + a_2}{2} (\Delta h), \text{ sometimes the formula}$$

$$\Delta S = \frac{\Delta h}{3} [a_1 + a_2 + \sqrt{a_1 a_2}] \text{ can also be used, where}$$

a_1, a_2, \dots represent the areas at 0.5 m contour interval.

Prismoidal formula can also be used preferably where three consecutive sections at equal height are taken. According to this,

$$\Delta S = \text{Storage} = \frac{\Delta h}{6} [A_1 + 4A_2 + A_3] \quad \dots (18.2)$$

where A_1, A_2 and A_3 are the areas of succeeding contours, and Δh is the vertical distance between two alternate contours.

It thus becomes evident that the areas at different elevations (contours), as well as the storage at different elevations, can be mathematically worked out, and both plotted on a graph paper, to obtain **Area-Elevation curve**, and **Storage-Elevation curve** (**Capacity-Elevation curve**), respectively, as shown in Fig. 18.3.

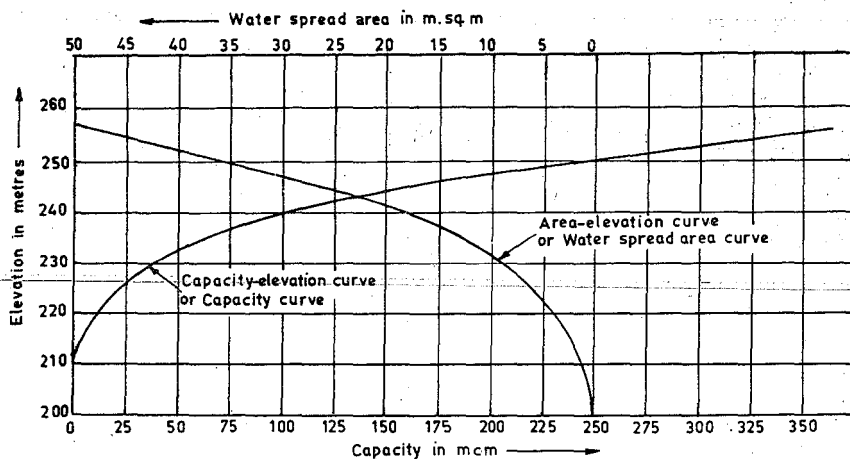


Fig. 18.3. Area elevation curve and Capacity elevation curve.

Another method for computing capacity may be the use of integration technique. Infact, the area-elevation curve, when integrated, will yield nothing but the capacity-elevation curve. Hence the surveyed areas at large-intervals, may be plotted on a simple graph paper, and a smooth curve, i.e. area-elevation curve, is first of all drawn. The equation of this curve is now obtained by statistical methods, which can be integrated to obtain the equation of the capacity-elevation curve, as follows:

The equation of the area-elevation curve, will generally be of the form:

$$A = \alpha + \beta \cdot h + \gamma \cdot h^2 + \dots \eta \cdot h^{n-1} \quad \dots(18.3)$$

where A represents the area at any elevation h ; and $\alpha, \beta, \gamma, \dots \eta$ are all constants.

This equation can be determined and then integrated to obtain the storage (capacity), as explained below:

Let y be the height of the water surface in the reservoir above any assumed datum, over which the storage/capacity is to be worked out.

Let A_y represents symbolically the area of the contour at this height. Then, assume that the equation of area-elevation curve is given by :

$$A_y = \alpha + \beta \cdot y + \gamma \cdot y^2 + \dots \eta \cdot y^{n-1} \quad \dots(18.3 a)$$

where $\alpha, \beta, \gamma, \dots, \eta$ are all constants.

From the actual survey, or from the points falling on the area-elevation curve, the area of any required number of contours (n) are known.

Let them be A_0, A_1, A_2, \dots corresponding to the known level values, i.e., heights $0, y_1, y_2, \dots$ above the datum.

Substituting these values in equation (18.3 a), we get

$$A_0 = \alpha$$

$$A_1 = \alpha + \beta \cdot y_1 + \gamma \cdot y_1^2 + \dots \eta \cdot y_1^{n-1}$$

$$A_2 = \alpha + \beta \cdot y_2 + \gamma \cdot y_2^2 + \dots \eta \cdot y_2^{n-1}$$

$$A_3 = \alpha + \beta \cdot y_3 + \gamma \cdot y_3^2 + \dots \eta \cdot y_3^{n-1}$$

Thus, we get n simultaneous equations to determine n number of constants ($\alpha, \beta, \gamma, \dots, \eta$) Hence, the equation of the area-elevation curve, i.e. $A_y = \alpha + \beta \cdot y + \gamma \cdot y^2 + \dots \eta \cdot y^{n-1}$ becomes defined, with $\alpha, \beta, \gamma, \dots \eta$ all known.

This equation can now be integrated between the limits 0 to y ,

$$\int_{y=0}^{y=y} A_y \cdot dy = \int_{y=0}^{y=y} (\alpha + \beta \cdot y + \gamma \cdot y^2 + \dots \eta \cdot y^{n-1}) dy$$

or S_y = storage (capacity) between 0 to y

$$= \int_{y=0}^{y=y} (\alpha + \beta \cdot y + \gamma \cdot y^2 + \dots \eta \cdot y^{n-1}) dy$$

$$= \left[\alpha \cdot y + \beta \cdot \frac{y^2}{2} + \gamma \cdot \frac{y^3}{3} + \dots + \eta \cdot \frac{y^n}{n} \right] + K$$

where K is a constant, which obviously is the reservoir capacity at datum.

The use of this method can be best understood by solving a numerical example.

Example 18.1. A contour survey of a reservoir site gives the following data:

Contour value	Area
At 200 m contour	6.0 hectares
At 210 m contour	18.1 hectares
At 220 m contour	34.0 hectares

The capacity of the reservoir upto 200 m elevation is found to be 14.1 ha. m. Determine the general equation for the area-elevation curve and capacity-elevation curve. Also determine the reservoir capacity at RL 225 m.

Solution. Use equation (18.3 a), wherein y is the height above RL 200 m, as

$$A_y = \alpha + \beta \cdot y + \gamma \cdot y^2$$

Now substituting the given values, we have

$$A_0 = 6.0 \text{ hectares}$$

$$\therefore 6.0 = \alpha + \beta (0) + \gamma (0) = \alpha$$

$$\text{or } \alpha = 6.0 \quad \dots(i)$$

$$\text{Also } A_1 = 18.1 \text{ hectares, at } y_1 = 10 \text{ m (given)}$$

$$\therefore 18.1 = \alpha + \beta \cdot (10) + \gamma \cdot (10)^2$$

$$\text{or } 18.1 = 6.0 + 10\beta + 100\gamma$$

$$\text{or } 12.1 = 10\beta + 100\gamma$$

$$\text{or } \beta + 10\gamma = 1.21 \quad \dots(ii)$$

$$\text{Also } A_2 = 34.0 \text{ hectares at } y_2 = 20 \text{ m (given)}$$

$$\therefore 34.0 = \alpha + \beta (20) + \gamma (20)^2 = 6.0 + 20\beta + 400\gamma$$

$$\text{or } 20\beta + 400\gamma = 28.0$$

$$\beta + 20\gamma = 1.4 \quad \dots(iii)$$

Solving (ii) and (iii), we get

$$\beta + 10\gamma = 1.21$$

$$\beta + 20\gamma = 1.40$$

$$\text{or } 10\gamma = 0.19 \quad \text{or } \gamma = 0.019$$

$$\therefore \beta + 0.19 = 1.21 \quad \text{or } \beta = 1.02$$

Hence, the equation of area-elevation curve is given by

$$A_y = 6.0 + 1.02y + 0.019y^2 \quad \text{Ans.}$$

Integrating this equation, we get

$$S_y = 6.0y + 1.02 \frac{y^2}{2} + 0.019 \frac{y^3}{3} + K$$

The constant K is obviously the reservoir capacity (storage) upto RL 200.0 m, which is given to be 14.1 ha. m.

$$\therefore S_y = 6.0y + 0.51y^2 + 0.0063y^3 + 14.1$$

Hence, the capacity-elevation curve is given by

$$S_y = 0.0063y^3 + 0.51y^2 + 6.0y + 14.1 \quad \text{Ans}$$

To determine the capacity at RL 225 m, we have to substitute

$y = 225 - 200 = 25$ m in the above eqn. to obtain the requisite capacity in ha.m., i.e.

Required capacity at RL 225 m

$$= 0.0063 (25)^3 + 0.51 (25)^2 + 6.0 \times 25 + 14.1$$

$$= 98.96 + 318.75 + 150.0 + 14.1 = 581.81 \text{ ha.m.} \quad \text{Ans.}$$

Conclusions. It can thus be seen that the capacity of a reservoir by this method can be determined by surveying only a few contours. It is also found that the method does not give more than 3% error, when it is cross-checked with the capacity worked out by surveying large number of contours. This error is not considered much, in the light of the fact that the areas of contours are themselves not very precise figures.

Sometimes, storage capacity may be expressed as a single term function of y , such as

$$S_y = K.y^n \quad \text{where } K \text{ and } n \text{ are constants.}$$

In fact, in practical life, no one bothers about the equations, and only curves, as shown in Fig. 18.3 are drawn. The required capacity at any elevation is read out, from such a curve.

It may also be pointed out here that the best capacity curve for a reservoir is the one in which the rise to the straight line is the quickest. It results from a cup-shaped catchment, with gentle longitudinal slope. Fig. 18.4 shows the good and bad capacity curves.

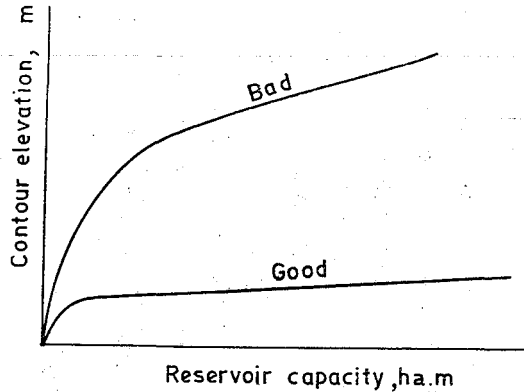


Fig. 18.4. Good and Bad capacity curves for a reservoir

18.3. Storage Zones of a Reservoir

These zones are defined w.r. to Fig. 18.5.

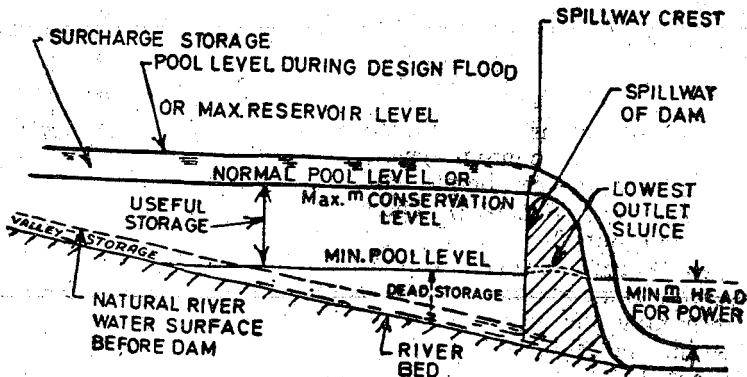


Fig. 18.5. Storage zones of a reservoir.

18.3.1. Normal Pool Level or Maximum Conservation Level. It is the maximum elevation to which the reservoir water surface will rise during normal operating conditions. (See Fig. 18.5). It is equivalent to the elevation of the spillway crest or the top of the spillway gates, for most of the cases.

18.3.2. Minimum Pool Level. The lowest water surface elevation, which has to be kept under normal operating conditions in a reservoir, is called the minimum pool level (See Fig. 18.5). This level may be fixed by the elevation of the lowest outlet in the dam or may be guided by the minimum head required for efficient functioning of turbines.

18.3.3. Useful and Dead Storage. *The volume of water stored in a reservoir between the minimum pool and normal pool levels is called the useful storage.* Water stored in the reservoir below the minimum pool level is known as the *Dead Storage*, and it is not of much use in the operation of the reservoirs. The useful storage may be subdivided into conservation storage and flood-mitigation storage, in a multipurpose reservoir.

18.3.4. Maximum Pool Level or Full Reservoir Level. During high floods, water is discharged over the spillway, but will cause the water level to rise in the reservoir above the normal pool level. The maximum level to which the water rises during the worst design flood is known as the *maximum pool level*.

18.3.5. Surcharge Storage. The volume of water stored between the normal pool level and the maximum pool level is called *surcharge storage*. Surcharge storage is an uncontrolled storage, in the sense that it exists only till the flood is in progress and cannot be retained for later use.

18.3.6. Bank Storage. When the reservoir is filled up, certain amount of water seeps into the permeable reservoir banks. This water comes out as soon as the reservoir gets depleted. This volume of water is known as the *bank storage*, and may amount to several percent of the reservoir volume depending upon the geological formations. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation capacity curve of the reservoir.

18.3.7. Valley Storage. Even before a dam is constructed, certain variable amount of water is stored in the stream channel, called *valley storage*. After the reservoir is formed, the storage increases, and the actual net increase in the storage is equal to the storage capacity of the reservoir minus the natural valley storage. The valley storage thus reduces the effective storage capacity of a reservoir. It is not of much importance in conservation reservoirs, but the available storage for flood mitigation is reduced, as given by the following relation:

$$\begin{aligned} \text{Effective storage for flood mitigation} \\ &= \text{Useful Storage} + \text{Surcharge Storage} - \text{Valley} \\ &\quad \text{Storage corresponding to the rate of inflow in the} \\ &\quad \text{reservoir.} \end{aligned}$$

DESIGNING RESERVOIR CAPACITY

18.4. Catchment Yield and Reservoir Yield

Long range runoff from a catchment is known as the yield of the catchment. Generally, a period of one year is considered for determining the yield value. *The total yearly runoff, expressed as the volume of water entering/passing the outlet point of the catchment, is thus known as the catchment yield, and is expressed in $M m^3$ or $M.ha.m$.*

The annual yield of the catchment upto the site of a reservoir, located at the given point along a river, will thus indicate the quantum of water that will annually enter the reservoir, and will thus help in designing the capacity of the reservoir. This will also help to fix the outflows from the reservoir, since the outflows are dependent upon the inflows and the reservoir losses.

The amount of water that can be drawn from a reservoir, in any specified time interval, called the *reservoir yield*, naturally depends upon the inflow into the reservoir and the reservoir losses, consisting of reservoir leakage and reservoir evaporation.

The annual inflow to the reservoir, i.e. the *catchment yield*, is represented by the *mass curve of inflow*; whereas, the outflow from the reservoir, called the *reservoir yield*, is represented by the *mass demand line* or the *mass curve of outflow*. Both these curves decide the reservoir capacity, provided the reservoir losses are ignored or separately accounted.

The inflows to the reservoir are however, quite susceptible of variation in different years, and may therefore vary throughout the prospective life of the reservoir. *The past available data of rainfall or runoff in the catchment is therefore used to work out the optimum value of the catchment yield.* Say for example, in the past available records of say 35 years, the minimum yield from the catchment in the worst rainfall year may be as low as say 100 M.ha.m; whereas, the maximum yield in the best rainfall year may be as high as say 200 M.ha.m. The question which then arises would be as to whether the reservoir capacity should correspond to 100 M. ha.m yield or 200 M. ha.m yield. If the reservoir capacity is provided corresponding to 100 M. ha.m yield, then eventually the reservoir will be filled up every year with a dependability of 100%; but if the capacity is provided corresponding to 200 M.ha.m yield, then eventually the reservoir will be filled up only in the best rainfall year (i.e. once in 35 years) with a dependability of about $\frac{1}{35} \times 100 \approx 3\%$.

In order to obtain a sweet agreement, a *via media* is generally adopted and an intermediate *dependability percentage value* (p), such as 50% to 75%,* may be used to compute the **dependable yield** or the **design yield**. The yield which corresponds to the worst or the most critical year on record is however, called the **firm yield** or the **safe yield**. Water available in excess of the firm yield during years of higher inflows, is designated as the **secondary yield**. Hydropower may be developed from such secondary water, and sold to the industries '*on and when available basis*'. The power commitments to domestic consumers must, however, be based on the firm basis, and should not exceed the power which can be produced with the firm yield, unless thermal power is also available to support the hydroelectric power.

The arithmetic average of the firm yield and the secondary yield is called the **average yield**.

18.4.1. Computing the Design or the Dependable Catchment Yield. The dependable yield, corresponding to a given dependability percentage p , is determined from the past available data of the last 35 years or so. The yearly rainfall data of the reservoir catchment is generally used for this purpose, since such long runoff data is rarely available. The rainfall data of the past years is therefore used to work out the dependable rainfall value corresponding to the given dependability percentage p . This dependable rainfall value is then converted into the dependable runoff value by using the available empirical formulas connecting the yearly rainfall with the yearly runoff.

It is, however, an adopted practice in Irrigation Departments to plan the reservoir project by computing the dependable yield from the rainfall data, but to start river gauging as soon as the site for the reservoir is decided, and then correlate the rainfall-runoff observations to verify the correctness of the assumed empirical relation between the rainfall and the runoff. Sometimes, on the basis of such observations, the initially assumed yield value may have to be revised.

* This percentage will depend upon the risk which can be absorbed for the proposed use of water. Say for example, city water supply projects can absorb lesser risk as compared to the irrigation projects, and hence should consider higher dependability percentage value.

The procedure which is adopted to compute the dependable rainfall value for a given dependability percentage p is explained below, and has been further used in solving example 18.2.

(i) The available rainfall data of the past N years is first of all arranged in the descending order of magnitude.

(ii) The order number m , given by the equation

$$m = N \cdot \frac{p}{100} \quad \dots(18.4)$$

is then computed, and rainfall value corresponding to this order number in the tabulated data will represent the required dependable rainfall value.

(iii) If the computed value of m is a fraction, then the arithmetic mean of the rainfall values corresponding to whole number m values above and below this fraction value is taken as the **dependable rainfall** value.

This method of computing the rainfall value of the given dependability (such as 50%, 75%, etc.) will become more clear when we solve example 18.2. The rainfall value of given dependability can finally be converted into the dependable yield, by using empirical methods, discussed in article 18.4.1.1.

The dependable yield of a given dependability (p per cent) can also be determined from the stream gauging, by computing the annual yields, from the observed discharges, and arranging the annual yield values in descending order to determine order numbers (instead of rainfall) as exhibited in solved example 18.3.

Example 18.2. *The yearly rainfall data for the catchment of a proposed reservoir site for 35 years is given in Table 18.1. Compute from this data, the values of dependable rainfalls for 60% and 75% dependability percentage.*

Table 18.1

Year	Rainfall in cm	Year	Rainfall in cm
1956	98	1974	88
1957	100	1975	94
1958	101	1976	107
1959	99	1977	110
1960	85	1978	208
1961	112	1979	114
1962	116	1980	104
1963	78	1981	120
1964	160	1982	108
1965	66	1983	102
1966	184	1984	80
1967	90	1985	109
1968	76	1986	122
1969	118	1987	115
1970	86	1988	140
1971	92	1989	138
1972	96	1990	60
1973	93		

Solution. The given data of table 18.1 is arranged in descending order, mentioning serial number (order number m) in front of each, as shown in Table 18.2.

Table 18.2

S.No. i.e. order number (m)	Rainfall in descending order in cm	S. No. i.e. order number (m)	Rainfall in descending order in cm
1	208	19	101
2	184	20	100
3	160	21	99
4	140	22	98
5	138	23	96
6	122	24	94
7	120	25	93
8	118	26	92
9	116	27	90
10	115	28	88
11	114	29	86
12	112	30	85
13	110	31	80
14	109	32	78
15	108	33	76
16	107	34	66
17	104	35	60
18	102		

Now, using equation (18.4), we compute the order number (m) for the given dependability percentage $p = 60\%$, as:

$$m = N \cdot \frac{p}{100} = 35 \times \frac{60}{100} = 21$$

The rainfall value tabulated above in table 18.2 at order no.21 is 99 cm; and hence the required dependable rainfall = 99cm. $\therefore P_{60\%} = 99$ cm Ans.

Similarly, for dependability $p = 75\%$, we calculate the order no. (m)

$$= N \cdot \frac{p}{100} = 35 \times \frac{75}{100} = 26.25$$

Since reqd. order no. is not an integer, the mean value of rainfall corresponding to order no. 26 and 27 will be taken as the value of p of 75% dependability.

$$\therefore P_{75\%} = \frac{92 + 90}{2} = 91 \text{ cm Ans.}$$

Example 18.3. The daily flows in a river for three consecutive years are given in the table by class interval alongwith the number of days the flow belonged to this class. What are the 50% and 75% dependable flows (annual and daily) for the river ?

Table 18.3

S. No.	Daily mean discharge m^3/s (range)	Year 1981	Year 1982	Year 1983
		No. of days the flow belonged to the given range (class interval)	No. of days the flow equalled the class range of col (2)	No. of days the flow equalled the range given in col (2)
(1)	(2)	(3)	(4)	(5)
1	100-90.1	0	6	10
2	90-80.1	16	19	16
3	80-70.1	27	25	38
4	70-60.1	21	60	67
5	60-50.1	43	51	58
6	50-40.1	59	38	38
7	40-30.1	64	29	70
8	30-20.1	22	48	29
9	20-10.1	59	63	26
10	10-negligible	54	26	13

Solution. With the given daily flows, we will calculate the annual yields for the given 3 years of records, as computed in Table 18.4. Since daily discharge figs. are given in range (interval), mid value of range (interval) can be assumed to be the flow discharge, which when multiplied by no. of days, will give yield during those days. Summation of yield values over 365 days will give annual yield, as shown in Table 18.4, which otherwise is self explanatory.

Table 18.4

S. No.	Daily mean discharge (range) (m^3/s) in descending order	Mean value of daily discharge range	Year 1981		Year 1982		Year 1983	
			No. of days for which the flow equalled to that given in col (3)	Yield (Mm^3) = (3) \times (4) $\times \frac{86400}{10^6}$ [(3) \times (4) \times 0.0864]	No. of days for which flow equalled to that of col (3)	Yield (Mm^3) = 0.0864 \times col (3) \times col (6)	No. of days for which flow equalled to that of col (3)	Yield (Mm^3) = 0.0864 \times col (3) \times col (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	100-90.1	95.05	0	0	6	49.25	10	82.08
2	90-80.1	85.05	16	117.50	19	139.54	16	117.50
3	80-70.1	75.05	27	174.96	25	162.00	38	246.24
4	70-60.1	65.05	21	117.94	60	336.96	67	376.27
5	60-50.1	55.05	43	204.34	51	242.35	58	275.62
6	50-40.1	45.05	59	229.39	38	147.77	38	147.74
7	40-30.1	35.05	64	193.54	29	87.70	70	211.68
8	30-20.1	25.05	22	47.52	48	103.68	29	62.64
9	20-10.1	15.05	59	74.34	63	81.65	26	33.70
10	10-Negligible	5.05	54	23.33	26	11.23	13	5.62
	Σ		365	1182.86	365	1361.98	365	1559.09

The river yield for 3 year is thus, calculated by summation of col (5), (7) & (9). Let these yields be arranged in descending order.

Year	Annual yield (Mm^3)	Order No. (m)
1983	1559.09	1
1982	1361.98	2
1981	1182.86	3
		$N=3$

Order No. (m) for $p\%$ dependability

$$= \frac{p}{100} \times N \quad \dots(\text{Eqn. 18.4})$$

(1) Order No for 75% dependability = $\frac{75}{100} \times 3 = 2.25$.

Since order No. of 2.25 is not an integer, the mean value of yield corresponding to S. NO. 2 and 3 will be taken as yield for 75% dependability, given as

$$= \left[\frac{1361.98 + 1182.86}{2} \right] = 1272.42 \text{ } Mm^3$$

Hence, Annual flow with 75% dependability = **1272.42 Mm^3** **Ans.**

\therefore Daily flow (discharge) with 75% dependability

$$= \frac{1272.42 \times 10^6}{365 \times 86400} \text{ } m^3/s = 40.34^3 \text{ } m^3/s \quad \text{Ans.}$$

(ii) Order No. for 50% dependability = $\frac{p}{100} \times N = \frac{50}{100} \times 3 = 1.5$

Since order No. of 1.5 is not an integer, the mean value of yield corresponding to order No. 1 and 2 shall be taken as the yield of 50%. Dependability, given as :

$$= \frac{1559.09 + 1361.98}{2} = 1460.54 \text{ M m}^3$$

Hence, Annual flow with 50% dependability = **1460.54 M m³** Ans.

∴ Daily flow (discharge) with 50% dependability

$$= \frac{1460.54 \times 10^6}{365 \times 86400} \text{ m}^3/\text{s} = \mathbf{46.31 \text{ m}^3/\text{s}}$$
 Ans.

18.4.1.1. Converting the dependable rainfall value into the dependable yield value. Certain empirical relations are available for converting the yearly rainfall value for the given catchment into the yearly runoff value expected from that catchment. Some of these formulas are:

1. *Binnie's percentages;*
2. *Strange's tables;*
3. *Barlow's tables;*
4. *Lacey's formula;*
5. *Inglis formula; and*
6. *Khosla's formula.*

These empirical relations are briefly discussed below:

(1) **Binnie's percentages.** The first effort ever made in India to connect the long range rainfall and the runoff (yield), was from Sir Alexander Binnie. He made observations *on two rivers in the central provinces*, and worked out certain percentages to connect the monthly rainfall with the monthly yield for the entire monsoon period from June to October. These percentages have been further adjusted by Mr. Garret, and are given in table 18.5. From the values given in table 18.5, *the total monsoon rainfall Vs percentage of rainfall that becomes runoff* are plotted in Fig. 18.6(a).

Table 18.5. Binnie's Monsoon Yield Percentages (as adjusted by Garret)

Monsoon Rainfall (P) in cm	Monsoon Yield percentage	Monsoon Rainfall (P) in cm	Monsoon Yield percentage
25	7.0	107.5	39.5
30	9.0	110.0	40.3
35	11.0	112.5	41.0
40	13.0	115.0	41.7
45	15.0	117.5	42.4
50	17.0	120.0	43.1
55	19.0	122.5	43.7
60	21.0	125.0	44.4
65	23.0	127.5	45.0
70	25.0	130.0	45.6
75	27.0	132.5	46.2
80	29.0	135.0	46.8
85	31.0	137.5	47.3
90	33.0	140.0	47.9
95	35.0	142.5	48.4
100	37.0	145.0	49.0
102.5	37.9	147.5	49.5
105.0	38.7	150.0	50.0

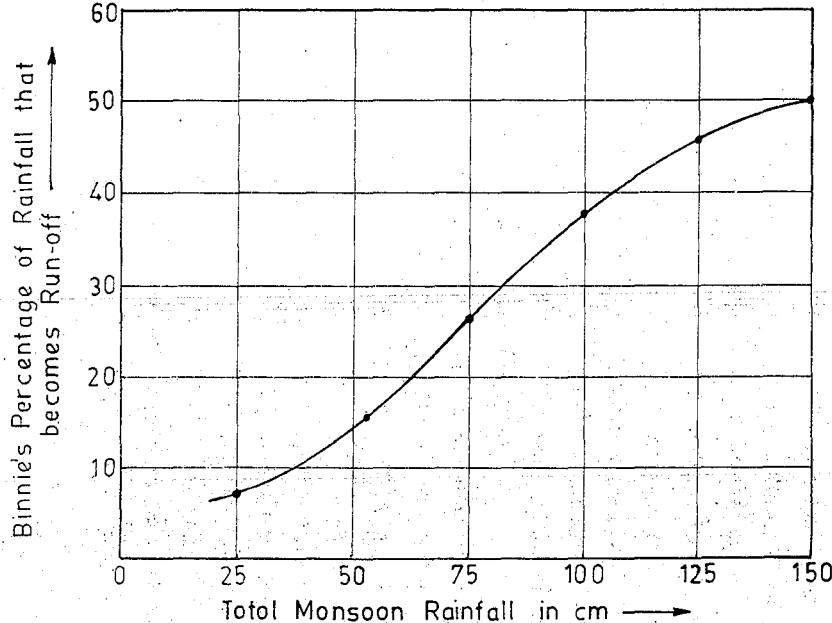


Fig. 18.6(a). Binnie's Monsoon Rainfall-runoff Curve.

(2) **Strange's percentages and tables.** Mr. W.L. Strange carried out investigations on catchments in Bombay Presidency, and worked out percentages for converting monsoon rainfall into monsoon yield. He even worked out such percentages for converting daily rainfalls into daily runoffs. It was an improvement over Binnie's tables, since he divided the catchments into three categories to account for the general characteristics of the catchments. The catchments prone to producing higher yields, such as those with more paved areas, etc. were categorised as *good catchments*; and those prone to producing low yields were termed as *bad catchments*. The intermediate types were called *average catchments*. Different runoff percentages were given for different types of catchments for different values of rainfalls, as shown in table 18.6(a). These values have also been drawn in the shape of curves, as shown in Fig. 18.6(b).

Table 18.6. Values of Strange's Run off Percentages

Monsoon Rainfall in cm	Runoff Percentages for catchments designated as			Monsoon Rainfall in cm	Runoff Percentages for catchments designated as		
	Good	Average	Bad		Good	Average	Bad
25.0	4.3	3.2	2.1	107.5	40.9	30.6	20.4
30.0	6.2	4.6	3.1	110.0	42.0	31.5	21.0
35.0	8.3	6.2	4.1	112.5	43.1	32.3	21.5
40.0	10.5	7.8	5.2	115.0	44.3	33.2	22.1
45.0	12.8	9.6	6.4	117.5	45.4	34.0	23.2
50.0	15.0	11.3	7.5	120.0	46.5	34.8	23.8
55.0	17.3	12.9	8.6	122.5	47.6	35.7	23.8
60.0	18.5	14.6	9.7	125.0	48.8	36.6	24.4
65.0	21.8	16.3	10.9	127.5	49.9	37.4	24.9
70.0	24.0	18.0	12.0	130.0	51.0	38.2	25.5
75.0	26.3	19.7	13.1	132.5	52.0	39.0	26.0
80.0	28.5	21.3	14.2	135.0	53.3	39.9	26.6
85.0	30.8	23.1	15.4	137.5	54.4	40.8	27.2
90.0	33.0	24.7	16.5	140.0	55.5	41.6	27.7
95.0	35.3	26.4	17.6	142.5	56.6	42.4	28.3
100.0	37.5	28.1	18.7	145.0	57.8	43.3	28.9
102.5	38.5	28.9	19.3	147.5	58.9	44.1	29.4
105.0	39.5	29.8	19.9	150.0	60.0	45.0	30.0

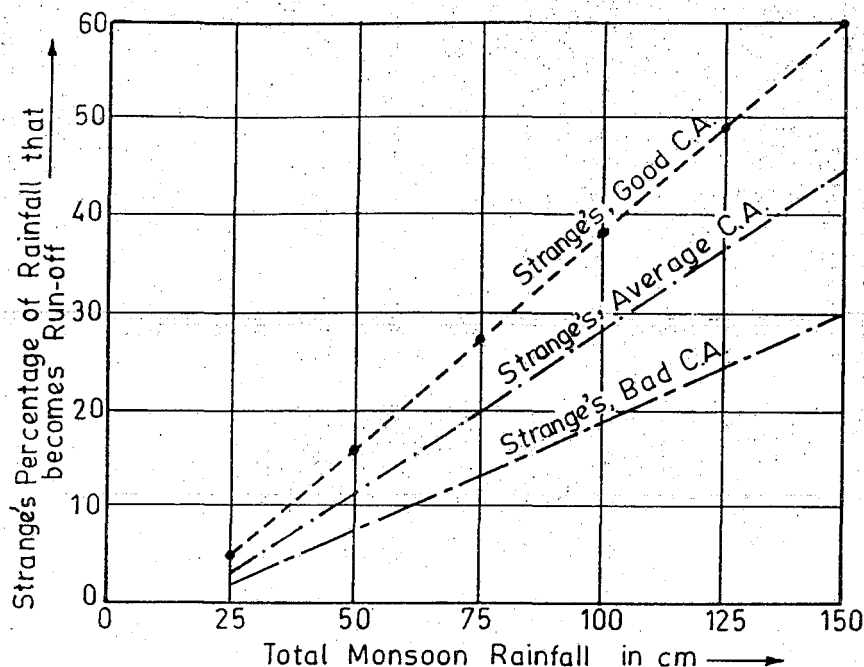


Fig. 18.6(b). Strange's monsoon Rainfall-Runoff curves.

(3) **Barlow's tables.** Mr. T.G. Barlow, the first Chief engineer of Hydro-Electric Survey of India, carried out extensive investigations on catchments mostly under 130 square km in U.P. State of India. On the basis of his investigations, he divided catchments into five classes and assigned different percentages of rainfall that become runoff (over long periods) for each class of catchment, as indicated in table 18.7.

Table 18.7. Values of Barlow's Runoff Percentages

S.No.	Class of catchment	Description of the catchment	Runoff percentages (K)
1.	A	Flat, cultivated, adsorbent soils	10
2.	B	Flat, partly cultivated stiff soils	15
3.	C	Average catchment	20
4.	D	Hills and plains with little cultivation	25
5.	E	Very hilly and steep catchment with little or no cultivation	33

The above values of runoff percentages are for the average type of monsoons, and are to be modified by the application of the following coefficients (Table 18.8) according to the nature of the season.

Table 18.8. Values of Barlow's coefficient to be multiplied with K values of table 18.7 to obtain True Runoff Percentages

S.No.	Nature of season	Class of catchment				
		A	B	C	D	E
1.	Light rain, no heavy down pour	0.7	0.8	0.8	0.8	0.8
2.	Average or varying rainfall, no continuous down pour	1.0	1.0	1.0	1.0	1.0
3.	Continuous down pour	1.5	1.5	1.6	1.7	1.8

(4) **Lacey's formula.** This formula connects the monsoon rainfall (P) with the yield (Q) by the equation.

$$\text{Yield } (Q) = \left[\frac{P}{1 + \frac{304.8 m}{p.n}} \right] \quad \dots(18.5)$$

where m = a constant, called *monsoon duration factor*, the values of which are given in table 18.9.

n = a constant, called *catchment factor*, the values of which are given for different classes of catchments (as defined by Barlow) in table 18.10.

Table 18.9. Values of m to be used in Eq. (18.5)

S.No.	Duration of Monsoon	Monsoon Duration factor m
1.	Bad year	0.5
2.	Normal year	1.2
3.	Good year	1.5

Table 18.10. Values of n to be used in Eq. (18.5)

S.No.	Class of catchment	Description of the catchment	Values of n
1.	A	Flat cultivated, absorbent soils	0.25
2.	B	Flat, partly cultivated stiff soils	0.60
3.	C	Average catchments	1.00
4.	D	Hills and plains with little cultivation	1.70
5.	E	Very hilly and steep catchments with little or no cultivation	3.45

(5) **Inglis formula.** Inglis derived his formula for catchments of West Maharashtra State of India. He divided the areas as ghat areas (*Sahyadri ranges*) where rainfall is 200 cm or more; and non-ghat areas where rainfall is less than 200 cm. His formulas are:

(a) For ghat areas, with rainfall (P) equalling or exceeding 200 cm:

$$\text{Yield} = (0.85P - 30.48) \text{ cm} \quad \dots(18.6)$$

where P is the rainfall in cm.

(b) For non-ghat areas with rainfall P less than 200 cm.

$$\text{Yield} = \frac{P(P - 17.78)}{254} \text{ cm} \quad \dots(18.7)$$

where P is the rainfall in cm.

(6) **Khosla's formula.** This formula is based upon the recent research work conducted in this field, and is a very simple and useful formula. It can be easily applied to the entire country, without bothering for the region of its origin. This formula states that

$$\text{Yield } (Q) = P - 0.48 T_m \quad \dots(18.8)$$

where Q = the yield in cm

P = the rainfall in cm

T_m = mean annual temperature of the area.

Example 18.4. The design annual rainfall for the catchment of a proposed reservoir has been computed to be 99 cm. The catchment area has been estimated to have the mean annual temperature of 20°C . The catchment area contributing to the proposed reservoir is 1000 sq.km. Calculate the annual design catchment yield for this reservoir. Make use of Khosla's formula.

Solution. Use Khosla's formula connecting the design rainfall (P) with the design yield (Q) by Eq. (18.8) as:

$$Q = P - 0.48 T_m$$

where $P = 99 \text{ cm}$ (given)

$T_m = 20^\circ\text{C}$ (given)

$$\therefore Q = 99 - 0.48 \times 20 = 89.4 \text{ cm} = \mathbf{0.894 \text{ m}}$$

The total yield produced in m^3 from the given catchment of 1000 sq.km (i.e. $1000 \times 10^6 \text{ m}^2$)

$$= 0.894 \times 1000 \times 10^6 \text{ m}^3 = 894 \text{ M.m}^3 \quad \text{Ans.}$$

18.4.2. Use of Flow Duration Curves for Computing Dependable Flow

As we know, the stream flow varies widely over a **water year***. This variability of stream flow can be studied by plotting **flow duration curves** for the given stream. A **flow duration curve**, also called as **discharge-frequency curve**, is a curve plotted between **stream flow (Q)** and **percent of time the flow is equalled or exceeded (P_p)**, and is of the type shown in Fig. 18.7 (a), (b) and (c).

Such a curve can be plotted by first arranging the stream flow values (Q) in descending order using class intervals, if the number of individual values is very large. The data to be used can be of daily values, weekly values, or monthly values. If N data

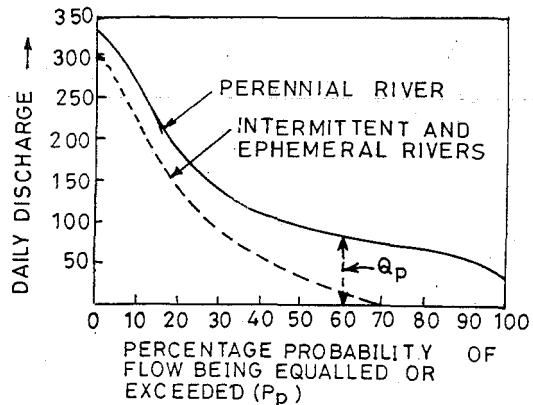


Fig. 18.7(a). Typical flow-duration curves on an ordinary paper.

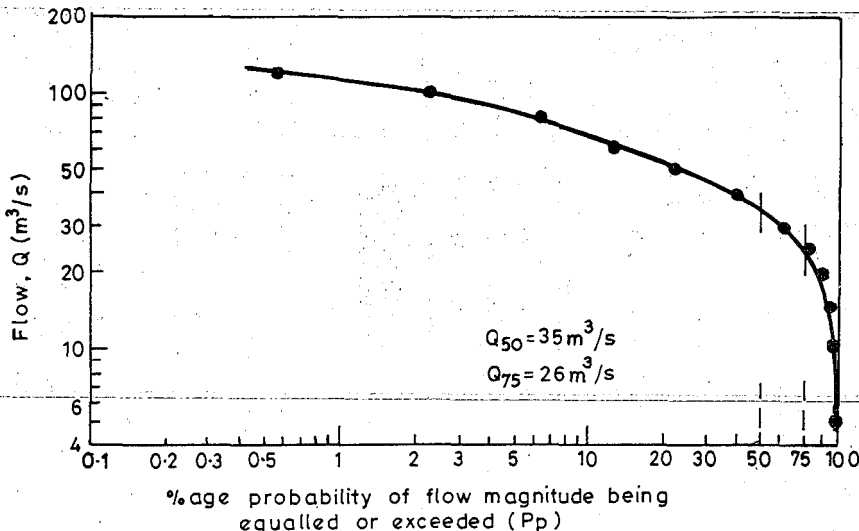


Fig. 18.7(b). A typical flow duration curve on a log-log paper.

values are used, the plotting position of any discharge (or class value) Q is given, as :

* 1st June to 30st May.

$$P_p = \frac{m}{N+1} \times 100\%$$

where m = is the order No. of that discharge (or class value)

P_p = percentage probability of the flow magnitude being equalled or exceeded.

The ordinate Q at any percentage probability p (such as 60%), i.e. Q_p , will represent the flow magnitude of the river that will be available for 60% of the year, and is hence termed as 60% dependable flow (Q_{60}). Q_{100} (i.e. 100% dependable flow) for a perennial river can, thus, be read out easily from such a curve. Q_{100} for an *ephemeral* or for an *intermittent river* shall evidently be zero.

A flow-duration curve represents the cumulative frequency distribution, and can be considered to represent the stream flow variation of an average year. Such a curve can be plotted on an ordinary arithmetic scale paper, or an semi-log or log-log paper. The following *characteristics of flow-duration curves* have been noticed.

(1) The slope of the flow-duration curve depends upon the interval of data used. Say for example, a daily stream flow data gives a steeper curve than a curve based on monthly data for the same river. This happens due to smoothening of small peaks in monthly data.

(2) The presence of a reservoir on a stream upstream of the gauging point will modify the flow-duration curve for the stream, depending upon the reservoir-regulation effect on the released discharges. A typical reservoir regulation effect on flow duration curve is shown in Fig. 18.7 (c).

(3) The flow-duration curve, when plotted on a log probability paper, is found to be a straight line at least over the central region, as shown in Fig. 18.7 (c). From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.

(4) The flow-duration curve plotted on a log-log paper (Fig. 18.7 (b)) is useful in comparing the flow characteristics of different streams. Say for example, a steep slope of the curve indicates a stream with a highly variable discharge; while a flat slope of the curve indicates a small variability of flow and also a slow response of the catchment to the rainfall. A flat portion on the lower end of the curve indicates considerable base flow. A flat portion on the upper end of the curve is typical of river basins having large flood plains, and also of rivers having large snowfall during a wet season.

(5) The chronological sequence of occurrence of the flow gets hidden in a flow duration curve. A discharge of say $500 \text{ m}^3/\text{s}$ in a stream will, thus, have the same percentage probability (P_p), irrespective of whether it occurred in January or June. This aspect, a *serious handicap of such curves*, must be kept in mind while interpreting a flow-duration curve.

Flow-duration curves find a considerable use in water-resources planning and development activities. Some of their important uses are indicated below:

- (i) For evaluating dependable flows of various percentages, such as 100%, 75%, 60%, etc. in the planning of water-resources engineering projects.

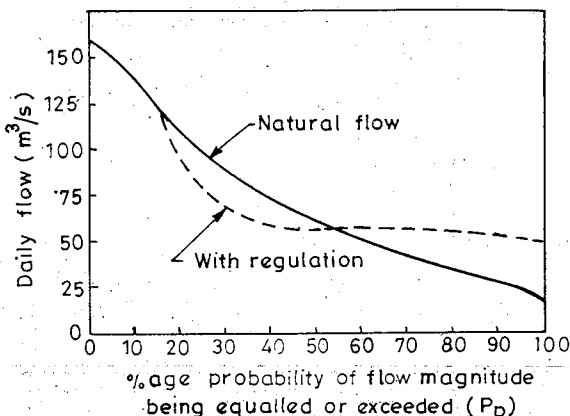


Fig. 18.7. (c) Reservoir regulation effect on the flow-duration curve of a stream.

- (ii) In evaluating the characteristics of the hydropower potential of a river.
- (iii) In comparing the adjacent catchments with a view to extend the stream flow data.
- (iv) In computing sediment load and dissolved solids load of a stream.
- (v) In the design of drainage systems, and
- (vi) in flood control studies.

18.5. Fixing the Reservoir Capacity for the Computed Value of the Dependable Yield of the Reservoir Catchment

After deciding the dependable yield for the proposed reservoir (tank), the reservoir capacity is decided as follows:

The water demand (annual of course) is computed by estimating the *crop water requirement* (including transit losses) and any other water demand required to meet the water supply needs, or the downstream commitments of water release, if any. Reservoir losses @ about 15% of the water demand is then added to obtain the *live* or the *net storage* required to meet the given demand. *Dead storage* is now added to this live storage to obtain the *gross storage* required to meet the demand. The *reservoir capacity*, however, cannot exceed the *catchment yield* (inflow into the reservoir), and hence the reservoir capacity is fixed at a value which is lesser of the value of the assessed gross storage required to meet the demand; and ii) the assessed dependable yield for the reservoir site. The full tank level (FTL) or the full reservoir level (FRL) is finally computed from the elevation-capacity curve.

The *dead storage* level or dead storage required in the above computation is usually fixed at higher of the following values:

- (a) dead storage = rate of silting \times Life of the reservoir
- (b) dead storage = 10% of gross storage or net water demand.
- (c) dead storage level being equal to the full supply level of the off taking canal at the tank site

Example 18.5. The lowest portion of the capacity-elevation curve of a proposed irrigation reservoir, draining 20 km^2 of catchment, is represented by the following data:

Elevation in m	Capacity in ha.m
RL 600	24.2
602	26.2
604	30.3
606	36.8

The rate of silting for the catchment has been assessed to be $300 \text{ m}^3/\text{km}^2/\text{year}$. Assuming the life of the reservoir to be 50 years, (a) compute the dead storage, and the lowest sill level (LSL), if the main canal is 6km long with a bed slope of 1 in 1000, and the canal bed level at the tail end is at RL 594.5 m. The FSD of the canal at the head is 80 cm. The crop water requirement is assessed as 250 ha.m.

(b) If the dependable yield of the catchment is estimated to be 0.29m, what will be the gross capacity of the reservoir ?

Solution. The dead storage is first of all computed as maximum of the following three values :

$$\begin{aligned}
 \text{(a) dead storage} &= \text{rate of silting} \times \text{life of the reservoir} \\
 &= 300 \text{ m}^3/\text{km}^2/\text{year} \times 20 \text{ km}^2 \times 50 \text{ years} = 300000 \text{ m}^3 \\
 &\quad \text{(C.A.)} \quad \text{(Life)} \\
 &= 30 \text{ ha.m.} \quad \dots(i)
 \end{aligned}$$

$$\begin{aligned}
 \text{(b) dead storage} &= 10\% \times \text{net water demand or crop water requirement} = 10\% \times 250 \text{ ha.m} \\
 &= 25 \text{ ha.m} \quad \dots(ii)
 \end{aligned}$$

(c) Dead Storage Level = FSL of canal at headworks

$$= 594.5 + (6 \times 1000) \frac{1}{1000} + 0.8 = 601.4 \text{ m}$$

Dead Storage Capacity at RL 601.4 m is interpolated

$$= 24.2 + \frac{(26.2 - 24.2) \text{ ham}}{RL (602 - 600)} \times RL (601.4 - 600) = 24.2 + \frac{2 \times 1.4}{2}$$

$$= 24.2 + 1.4 \text{ ham} = 25.6 \text{ ham} \quad \dots(iii)$$

The dead storage is fixed at maximum of the three values obtained at (i), (ii) and (iii) above, i.e. 30 ham, 25 ham, and 25.6 ham. Hence, choose the dead storage at **30 ham. Ans.**

The *LSL* (x) corresponds to 30 ham capacity, which is computed as :

$$30 = 26.2 \text{ ham} + \frac{(30.3 - 26.2) \text{ ham}}{RL (604 - 602)} \times RL (x - 602)$$

$$\text{or} \quad 3.8 = \frac{4.1}{2} (x - 602)$$

$$(x - 602) = \frac{3.8 \times 2}{4.1} = 1.85$$

$$x = \text{RL } 603.85 \text{ m.}$$

The lowest sill level i.e. dead storage level is thus fixed at **RL 603.85 m. Ans.**

(b) Net water demand = Crop water requirement including transit losses = 250 ham

Reservoir losses = $15\% \times 250 \text{ ham} = 37.5 \text{ ham}$

Live storage required

to meet the given demand = $(250 + 37.5) \text{ ham} = \mathbf{287.5 \text{ ham}}$

Dead storage = 30 ham (computed above)

\therefore Gross reservoir storage required

$$= \text{Live storage} + \text{Dead storage} = 287.5 + 30 = \mathbf{317.5 \text{ ham}}$$

Dependable yield = $0.29 \text{ m (depth)} = 0.29 \text{ m} \times (20 \times 10^6 \text{ m}^2) = \mathbf{580 \text{ ham}}$

The gross capacity of the reservoir is fixed at the lesser of the gross storage required to meet the demand (i.e. 317.5 ham) and the dependable yield (i.e. 580 ha.m.). Hence, the reservoir capacity = **317.5 ham. Ans.**

18.6. Relation between Inflow, Outflow, and storage Data for a Reservoir

The *inflow* to the reservoir and the *outflow* from the reservoir are the only two factors which govern the storage capacity of a reservoir. Since the inflow to the reservoir is variable, water is stored in the reservoir to cater to the required outflow from the reservoir, particularly during the critical periods in non-monsoon season. Naturally, if more outflow is required, more capacity has to be provided.

As a matter of fact, after assessing the monthly or annual inflows into the reservoir and representing it by the mass inflow curve, the demand pattern is specified. The reservoir is then usually designed to meet this specified demand, represented by the mass outflow curve.

The reservoir capacity, the reservoir inflow, and the outflow from the reservoir are governed by the storage equation, given by:

$$\text{Inflow} - \text{Outflow} = \text{Increase in storage}$$

$$\therefore \text{Increase in Reservoir Storage} = \text{Inflow} - \text{Outflow} \quad \dots(18.9)$$

18.7. Fixing the Reservoir Capacity from the Annual Inflow and Outflow Data

The capacity of reservoir may be determined by determining the storage needed to accommodate the given inflow minus the given outflow, as governed by the above equation. However, this study involves numerous factors, as discussed below:

Streamflow data at the reservoir site must be known. Monthly inflow rates are sufficient for large reservoirs, but daily data may be required for small reservoirs. When the inflow data at the dam site are not known, the data at a station elsewhere

on the stream or on a nearby stream may be collected and adjusted to the dam site. The available data may sometimes be extended so as to include a really drought period.

Besides determining the streamflow data at the dam site, an adjustment has to be made for the water required to be passed from the reservoir to satisfy the prior water rights and to obey the agreements between various sharing States through which the river is passing.

Moreover, the construction of a reservoir increases the exposed area of the water surface above that of the natural stream, and thus, increases the evaporation losses. There is, thus, a net loss of water occurring due to reservoir construction. Sometimes, these losses may be so huge that the entire purpose of the reservoir may be defeated. Seepage from the reservoir may also add to the loss resulting from the reservoir.

All these factors make this study very very complex. An approximate easy solution for determining the reservoir capacity may be obtained graphically with the help of *mass curves* as explained in the next article. On the other hand, a tabular solution is necessary in order to account for all important factors. For more precise results and for all complex systems, computers may be used for programming the analysis (called operation study).

Example 18.6. *Monthly inflow rates during a low-water period at the site of a proposed dam are tabulated in Col. (2) of table 18.9. The corresponding monthly pan evaporation and precipitation at a nearby station are also tabulated in Col. (3) and Col. (4) of the same table. Prior water rights make it obligatory to release the full natural flow or 15 hectare-metres per month, whichever is minimum. If the estimated monthly demands are as given in col. (5) of table 18.9 and the net increased pool area is 400 hectares, find the required storage capacity for the reservoir. Assume pan evaporation coefficient=0.7 and also assume that only 28% of the rainfall on the land area to be flooded by reservoir has reached the stream in the past.*

Table 18.11

Month	Inflow at dam site in hectare metres	Pan evaporation in cm	Precipitation in cm	Demand in hectare-metres
(1)	(2)	(3)	(4)	(5)
January	1.2	1.8	1.3	15.8
February	0.0	1.8	1.7	14.3
March	0.0	2.6	0.6	9.6
April	0.0	10.2	0.0	4.8
May	0.0	15.4	0.0	3.5
June	0.0	1.6	1.1	3.4
July	240.0	10.8	16.1	5.0
August	480.0	11.7	16.4	5.0
September	1.0	10.8	2.2	10.0
October	0.6	9.6	0.8	15.6
November	0.5	7.8	0.0	16.8
December	0.2	2.0	0.0	16.8
Σ	723.5	101.1	40.2	120.6

Solution. Table 18.11 is extended as shown in Table 18.12. The 28% precipitation is already reaching and is included in the given inflow [Col. (2)], and hence, only 72% of the precipitation is to be included, as worked out in Col. (8). The table is otherwise self-explanatory and the monthly water drawn from the reservoir is worked out in Col. (10).

Table 18.12. Solution Table for Example 18.6

Month	Inflow at dam site in hectare-metres	Pan evaporation in cm	Precipitation in cm	Demand in hectare-metres	Requirement due to prior rights equal to Col. (2) or 15 ha.m. whichever is minimum	Evaporation in hectare-metres $\frac{400 \times \text{Col. (3)}}{100} \times 0.7$ $= 2.8 \times \text{Col. (3)}$	Precipitation in hectare metres $\frac{400 \times \text{Col. (4)}}{100} \times 0.72$ $= 2.88 \times \text{Col. (4)}$	Adjusted inflow. Col. (2) + Col. (8) - Col. (6) - Col. (7) in hectare metres	Water required from storage in ha. m. Col. (5) - Col. (9) (only +ve values)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
January	1.2	1.8	1.3	15.8	1.2	5.03	3.74	(-) 1.29	17.09
February	0.0	1.8	1.7	14.3	0.0	5.03	4.9	(-) 0.13	14.43
March	0.0	2.6	0.6	9.6	0.0	7.28	1.73	(-) 5.55	15.15
April	0.0	10.2	0.0	4.8	0.0	28.56	0.0	(-) 28.56	33.36
May	0.0	15.4	0.0	3.5	0.0	43.1	0.0	(-) 43.1	46.60
June	0.0	16.6	1.1	3.4	0.0	46.4	3.17	(-) 43.23	46.63
July	240.0	10.8	16.1	5.0	15.0	30.3	46.4	(+) 241.1	Nil
August	480.0	11.7	16.4	5.0	15.0	32.8	47.3	(+) 479.5	Nil
September	1.0	10.8	2.2	10.0	1.0	30.3	6.34	(-) 23.96	33.96
October	0.6	9.6	0.8	15.6	0.6	26.9	2.31	(-) 24.39	39.99
November	0.5	7.8	0.0	16.8	0.5	21.8	0.0	(-) 21.8	38.60
December	0.2	2.0	0.0	16.8	0.2	5.6	0.0	(-) 5.6	22.40
Σ	723.5	101.1	40.2	120.6	33.5	283.1	116.49		308.21

Finally, their summation (i.e. 308.21 hectare-metres) works out to be the required storage capacity of the reservoir. **Ans.**

18.8. Fixation of Reservoir Capacity with the Help of Mass Curves of Inflow and Outflow

After the flow hydrographs for the stream at the dam site have been plotted for a large number of years (say 25 to 30 years), the required storage capacity for a reservoir

with a given outflow pattern can be approximately calculated with the help of mass curves. A hydrograph is a plot of discharge vs. time, while a *mass curve* is a plot of accumulated flow vs. time. The area under the hydrograph between times $t = 0$ and $t = t$ will represent nothing but the accumulated flow up to the time t , and hence, the ordinate of the mass curve at time t .

A typical annual inflow hydrograph is shown in Fig. 18.8 (a), and the mass curve for this inflow hydrograph is plotted in Fig. 18.8 (b). The area under the first curve upto a time (t) is equal to the ordinate of the second curve at the same time (t). Adjustments for the scales and units of the two curves must be made while plotting.

It is evident that a mass curve will continuously rise, as it is the plot of the accumulated inflow. Periods of no inflow would be represented by the horizontal lines on the mass inflow curve. To differentiate such a mass curve of runoff from the mass curve of rainfall, this mass curve is usually called as the **flow mass curve** and is an integral of the flow hydrograph.

The mass curve may also be called the *ripple diagram*. The slope of the mass curve at any time is a measure of the inflow rate at that time.

After the inflow mass curve has been plotted, the mass curve of demand may also be plotted by accumulating the required outflow. If a constant rate of withdrawal is required from the reservoir, the mass curve of demand will be a straight line having a slope equal to the demand rate. Demand curves or demand lines are generally straight lines (representing uniform withdrawal) although, in practice, they may be curved also [See Fig. 18.9 (a) and (b)].

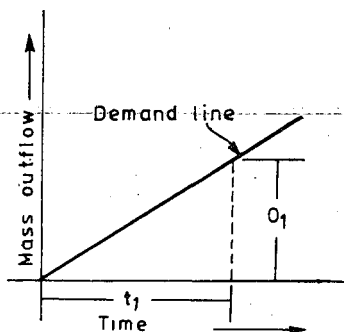


Fig. 18.9 (a)

$O_1/t_1 = \text{Slope of the line} = \text{Demand rate}$

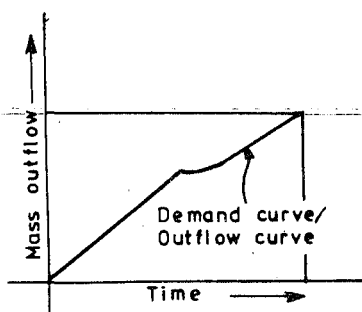


Fig. 18.9 (b).

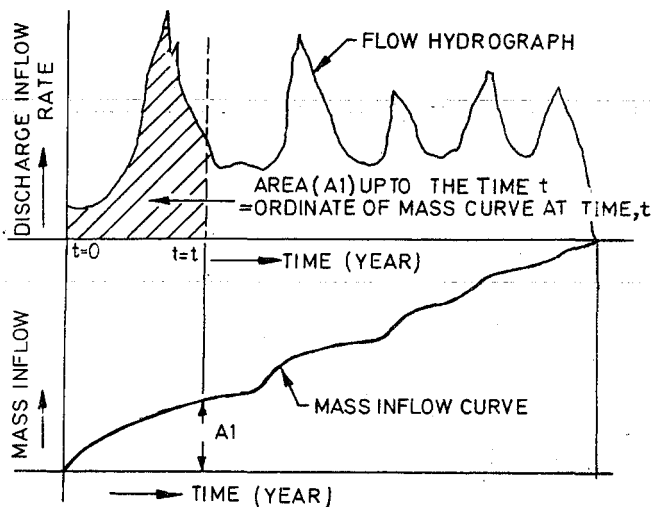


Fig. 18.8 (a) and (b).

Determining Reservoir Capacity for a Given Demand

The mass curve of inflow and the demand line can be used to determine the required storage capacity. In Fig. 18.10, it is evident that the demand lines drawn tangent to the high points $A_1, A_2, A_3 \dots$ of the mass curve, represent the rate of withdrawal from the reservoir. Assuming the reservoir to be full whenever the demand line intersects the mass curve (points $F_1, F_2 \dots$), the maximum departure ($B_1C_1, B_2C_2 \dots$) between the two curves represents the reservoir capacity just required to satisfy the demand.

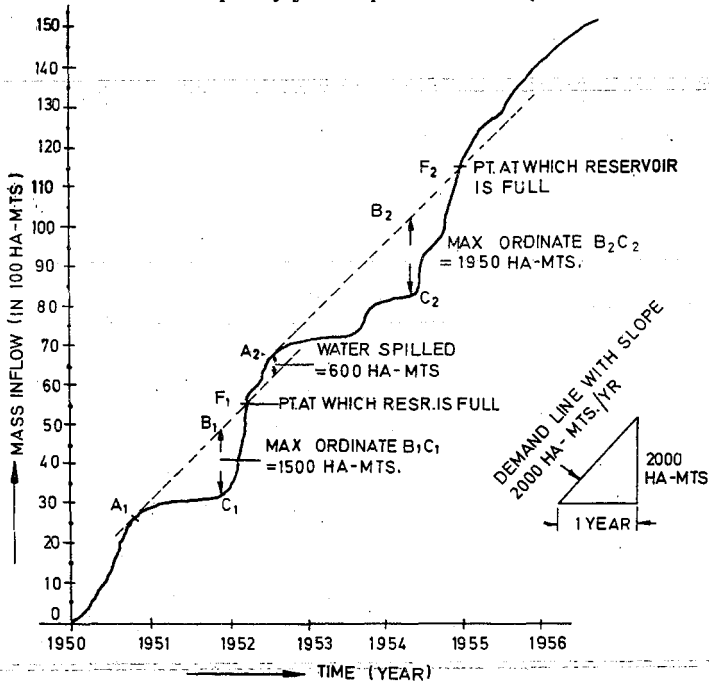


Fig. 18.10.

Fig. 18.10 has been drawn for a demand of 2,000 hectare- metres/year. The biggest departure ordinate (*i.e.* the maximum of $B_1C_1, B_2C_2 \dots$) works out to be 1,950 hectare- metres/year, which represents the required storage capacity for the reservoir.

The vertical distance between the successive tangents A_1B_1 , and A_2B_2 , etc., represent the water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume.

For Fig. 18.10, the spillway capacity works out to be 600 hectare- metres, and the reservoir capacity as 1,950 hectare- metres. It can also be observed that:

- (i) Assuming the reservoir to be full at A_1 , it is depleted to $1,950 - 1,500 = 450$ ha-m at C_1 and is again full at F_1 .
- (ii) The reservoir is full between F_1 and A_2 , and the quantity of water spilled over the spillway is equal to 600 ha-m.
- (iii) From A_2 , the water starts reducing in the reservoir till it becomes fully empty at C_2 .
- (iv) The water again starts collecting in the reservoir and it is again full at F_2 .

Note 1. It may also be noted that a demand line, when extended, must intersect the mass curve. If it does not, the reservoir will not refill.

Note 2. When the demand curve is not a straight line, then the two mass curves are superimposed over each other in such a way that their origins and axis coincide. The larger ordinate between the two, gives the required storage capacity, as shown in Fig. 18.11.

Fixing the Demand for a Reservoir of a Given Capacity

In the previous article, we have explained as to how the reservoir capacity can be determined for a given demand. The reverse, *i.e.* fixing the demand for a given reservoir capacity, may also be done

In this case, the tangents are drawn to the high points (A_1, A_2, \dots) of the mass inflow curve in such a way that the maximum departure from the mass curve is equal to the reservoir capacity. The slopes of the lines so drawn represent the demand rate which can be obtained with this capacity during different periods. The minimum value of these slopes will represent the withdrawal rate, which can certainly be obtained from the given reservoir, and will, thus, represent its *firm yield*.

For example in Fig. 18.12, a mass inflow curve is given. It is further required to find out the possible yields and the safe yield for a reservoir capacity of 750 ha-m. To determine these values, we shall proceed as follows :

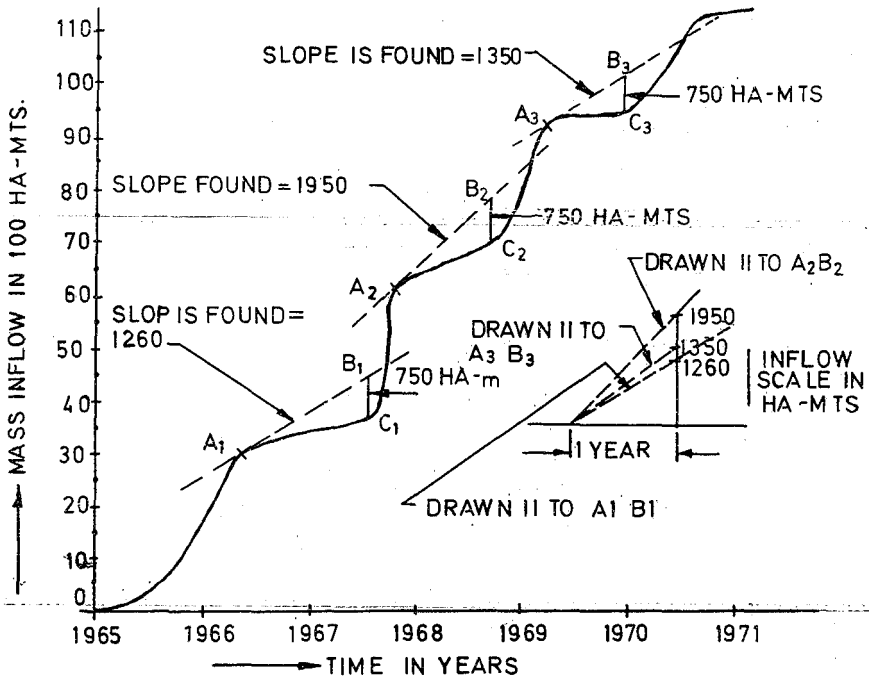


Fig. 18.12.

First of all, the high points A_1 , A_2 and A_3 are marked on the mass inflow curve. The points B_1 , B_2 , and B_3 are determined in such a way that their maximum departure from the curve is equal to 750 ha-m. The tangent lines A_1B_1 , A_2B_2 and A_3B_3 are then drawn. The slopes of these lines are determined and are found to be 1,260, 1,950, 1,350

ha-m/year, respectively. These values represent the possible yields in different periods. The minimum of them, i.e. 1,260 ha-m/year represents the safe yield or the firm yield of the reservoir. **Ans.**

Example 18.7. Annual runoff in terms of depth over the catchment area of 1675 sq.km. of a reservoir is given below :

Year	1962	63	64	65	66	67	68	69
Runoff (cm)	98	143.5	168.3	94	95.3	152.4	110	131.3

Draw the flow mass diagram. What is the average yield from the catchment? What should be the live storage capacity of the reservoir to use the source fully? If the dead storage is 20% of the live storage, what is the gross storage? Mark the filling and emptying periods on the mass curve. (Bhopal University, 1980)

Solution. The cumulative runoff values are worked out in Col. 4 of table 18.13, and they are plotted against the values of corresponding years (col. 1) of the same table, so as to obtain the desired mass diagram [Fig. 18.13].

Table 18.13

Year	Yearly runoff (cm)	Runoff as volume in $M.m^3 = \frac{\text{Col. (2)}}{100} \times 1675$	Cumulative runoff as volume in $M.m^3$
(1)	(2)	(3)	(4)
1962	98	1642	1642
63	143.5	2404	4046
64	168.3	2819	6865
65	94	1575	8440
66	95.3	1596	10,036
67	152.4	2553	12,589
68	110	1842	14,431
69	131.3	2199	16,630
	$\Sigma = 992.8$	16630	

The **average annual yield** of the catchment is the arithmetic mean of the given annual yields, and is equal to

$$\begin{aligned}
 &= \frac{992.8}{8} = 124.1 \text{ cm of runoff} \\
 &= 1.241 \text{ m} \times \text{CA in m}^2 \quad \text{(volume of runoff)} \\
 &= 1.241 \text{ m} \times (1675 \times 10^6) \text{ m}^2 \\
 &= 2078.68 \text{ Mm}^3; \text{ say } \mathbf{2079 \text{ Mm}^3} \quad \mathbf{Ans.}
 \end{aligned}$$

Now, to utilise the source fully, there should not be any spilling over of the water, and the yearly demand should be equal to the average yield i.e. 2079 Mm^3 .

To determine the required reservoir capacity to meet this demand rate, a line is drawn from the high point A_1 , parallel to this demand rate, as shown in Fig. 18.13; and the maximum departure of this line from the mass curve is read out as $B_1C_1 = 1065 \text{ Mm}^3$.

Hence, the required reservoir capacity is 1065 Mm^3 . **Ans.**

Physically speaking, the reservoir will be empty at trough pts C_1 and C_2 , and full at ridge point A_1 .

Alternatively, the mass demand line can be plotted, and both curves extended back up to the origin O , as shown in Fig. 18.14.

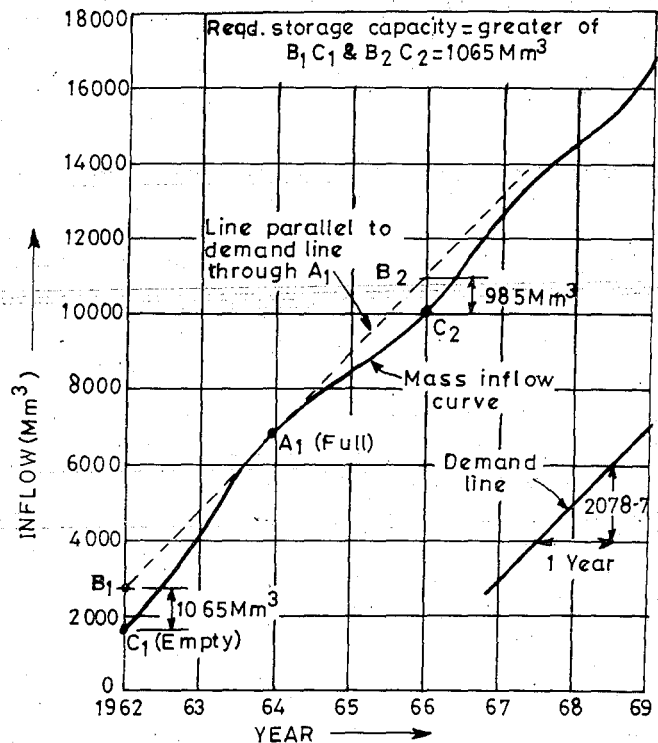


Fig. 18.13.

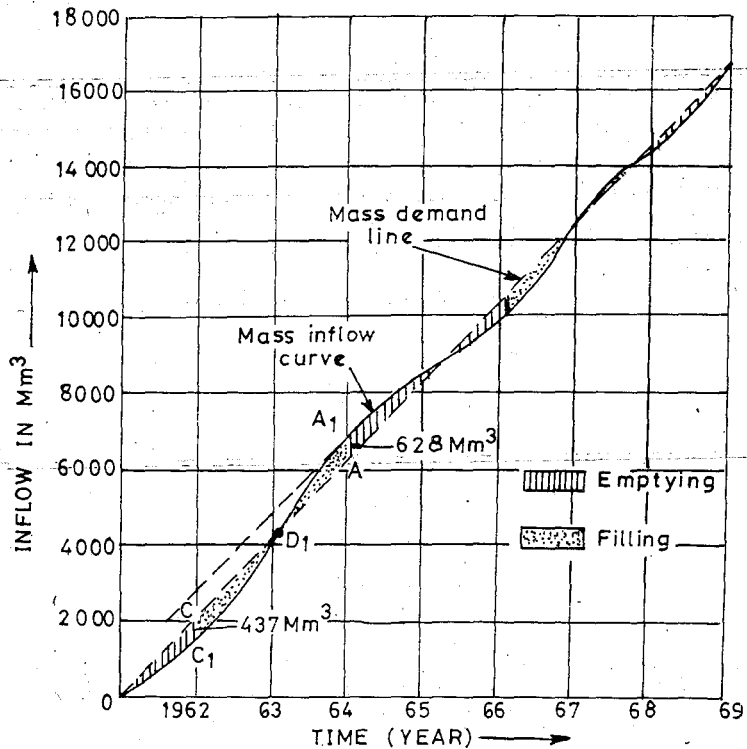


Fig. 18.14.

It can be seen from this curve, that from O to C_1 , the slope of the inflow curve is less than that of the demand curve, indicating that the inflow is less than the outflow, and the reservoir is *emptying*. From C_1 to D_1 , the slope of the inflow curve is in excess of the demand, and reservoir is *filling*. The vertical intercept C_1C , from the inflow curve at C_1 to the demand curve at C , represents the initial storage to meet the demand from O to C_1 . At D_1 , the reservoir level is the same as at O . After D_1 , and upto A_1 , the slope of the inflow curve is steeper than that of the demand curve, and as such, the reservoir is still rising. Point A_1 represents the full reservoir level. Vertical intercept A_1A represents the storage between the initial water level corresponding to the point O and the full reservoir level. The total minimum storage capacity of the reservoir to meet the demand is thus given by the vertical intercepts C_1C and A_1A .

$$\begin{aligned} \text{Max. withdrawal from storage} + \text{Max. stored in storage} \\ = 437 + 628 = 1065 \text{ mcm. } \text{Ans.} \end{aligned}$$

The emptying and filling processes are also shown in Fig. 18.14 ; filling will occur when the slope of the inflow curve is more than that of the demand curve, and *vice versa*.

18.9. Fixation of Reservoir Capacity Analytically using Sequent Peak Algorithm

The *sequent peak algorithm* is a simple and straight forward analytical procedure, for computing reservoir capacity, and is used as an excellent alternative to the mass curve method of determining reservoir capacity.

In the mass curve analysis, the reservoir is assumed to be full at the beginning of the dry period, and storage required to pass the dry period is estimated. If the mass curve contains only one ridge point, and if there is no well defined subsequent trough point, it may become necessary to repeat the given data for one more cycle to arrive at the desired storage determination. Also, the demand rate is usually not a straight line (as assumed in mass curve analysis), since the demand (out flow) generally becomes non-uniform due to seasonal variations in the demand. *The sequent peak algorithm technique helps us to devise simple mathematical solution to the problem of computing the reservoir capacity.*

Sequent peak algorithm (Fig. 18.15) is a plot between time (say, in months) on X-axis, and cumulative inflow minus cumulative outflow on Y-axis. The quantity taken on Y-axis, corresponding to each month equals to $[\Sigma \text{ Inflow} - \Sigma \text{ outflow}] = [\Sigma (\text{Inflow} - \text{Outflow})]$. This value is also called *cumulative net inflow*.

The +ve values of cumulative net inflow, representing cumulative surplus of inflow will be plotted above X-axis, while its negative values, representing cumulative deficit of inflow, will be plotted below x-axis.

The obtained plot will consist of peaks and troughs, as shown in Fig. 18.15.

The first ridge point A_1 in this plotting (*i.e.* first peak of cumulative net surplus) is called the *first peak* ; while subsequent ridge points A_2, A_3, A_4 , etc., are called *sequent peaks*.

Similarly, the first trough point B_1 is called the first trough, while subsequent trough points B_2, B_3 , etc. may be called the *sequent troughs*.

The difference between the first peak and first trough (height A_1B_1) will, in this plot, evidently, represent the reservoir storage required under normal inflows, and

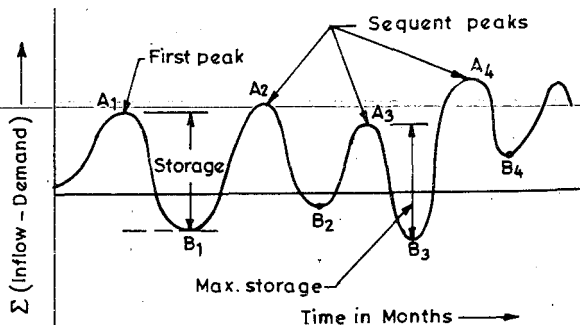


Fig. 18.15. Sequent Peak Algorithm.

is called the *normal storage* ; whereas, the maximum difference between any sequent peak and the just following trough will represent the *maximum storage* required for the reservoir.

The normal and maximum storage through sequent peak algorithm is calculated as follows :

- (1) Convert the monthly inflows into the volume units for the period of the available data.
- (2) Estimate the monthly volumes of all the outflows from the reservoir. This should include losses from evaporation, seepage, and other losses.
- (3) Compute the cumulative values of Inflows.
- (4) Compute the cumulative values of outflows.
- (5) Compute the values of cumulative inflow minus cumulative outflow ; i.e. $[\Sigma \text{ Inflow} - \Sigma \text{ Outflow}]$.
- (6) Plot a graph by taking months (time) on X-axis, and $\Sigma (I - O)$ of step (5) on Y-axis, on an ordinary graph paper.
- (7) The data will plot peaks and troughs. The second and subsequent peaks are called sequent peaks.
- (8) The maximum difference between any sequent peak and the just following trough is the maximum storage required for the reservoir. The difference between the first peak and the trough following it, is the storage required under normal inflows.

The method will become more clear on solving example 18.8.

Example 18.8. *Monthly inflows at a proposed reservoir site for a drought period of 15 months are given along with targetted demands (found from a working table) in the table below. Compute the storage required by plotting sequent peak algorithm.*

Table 18.14

Month	June	July	Aug	Sept	Oct	Nov	Dec	Jan	Feb	March	April	May	June	July	Aug
River Inflows ($M m^3$)	250	350	400	200	150	150	100	50	150	300	400	450	150	200	450
Targetted demand $M m^3$	150	150	200	250	350	400	250	200	150	150	100	250	350	300	100

Solution. Calculations are carried out in Table 18.15 to compute $\Sigma (I - O)$ in col (6).

Table 18.15

Month	Inflow (I) $M m^3$	Outflow (O) $M m^3$	Cumulative Inflow $\Sigma I (M m^3)$	Cumulative Outflow $\Sigma O (M m^3)$	$(\Sigma I - \Sigma O)$
(1)	(2)	(3)	(4)	(5)	(6)
June	250	150	250	150	+ 100
July	350	150	600	300	+ 300
Aug	400	200	1000	500	+ 500
Sept	200	250	1200	750	+ 450
Oct	150	350	1350	1100	+ 250
Nov	150	400	1500	1500	0
Dec	100	250	1600	1750	(-) 150
Jan	50	200	1650	1950	(-) 300
Feb	150	150	1800	2100	(-) 300
March	300	150	2100	2250	(-) 150
April	400	100	2500	2350	+ 150
May	450	250	2950	2600	+ 350
June	150	350	3100	2950	+ 150
july	200	300	3300	3250	+ 50
Aug	450	100	3750	3350	+ 400

To calculate values of $\Sigma (I - O)$, values of ΣI (cumulative inflow) are written in col (4), and values of ΣO (cumulative outflow) are written in col (5).

Values of $(\Sigma I - \Sigma O) = \Sigma (I - O)$ are now calculated by subtracting values of col (5) from values of col (4). These values are written in col (6) with their +ve or -ve sign, representing excess & cumulative deficit, respectively. When values of $(\Sigma I - \Sigma O)$ from col (6) are plotted on y-axis along with corresponding values of month (col 1) taken on x-axis, a plot containing peaks and troughs will be obtained, as shown in Fig. 18.16, called sequent peak algorithm. The difference between first peak and the trough following it, is found to be **800 M m³**, which represents the reqd. storage capacity. **Ans.**

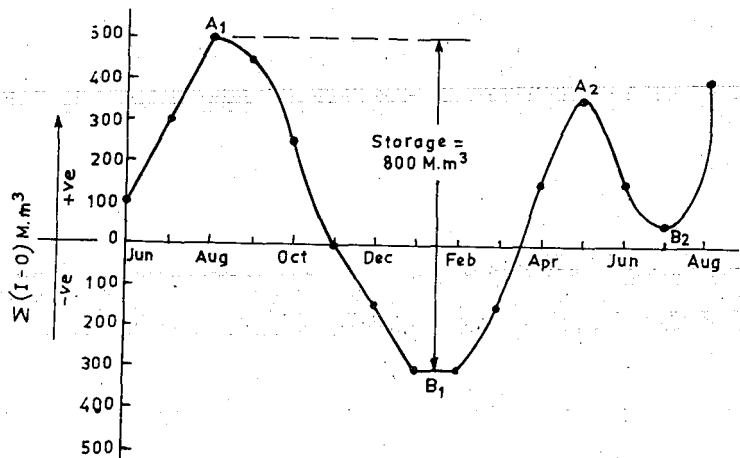


Fig. 18.16. Sequent peak algorithm for Example 18.8.

Alternatively, the values of $\Sigma (I - O)$ can be more easily computed by first computing +ve and -ve values of $(I - O)$, as in col (4) & (5) of Table 18.16 ; and then computing their summation values, i.e. values of $\Sigma (I - O)$ as in col (6) and (7) of Table 18.6. +ve values represent cumulative excess inflow ; while -ve values represent cumulative deficit. The maximum value out of all the values of Col (6) and (7) will represent the minimum storage required to accommodate the surplus (or to supply the deficit). This method thus, avoids the necessity of plotting peaks & troughs (curve) and the value of reqd storage capacity can be computed easily and mathematically, which procedure can even be programmed in a digital computer to help in estimation of required storage capacity. In present calculations, this peak value in col (6) & (7) is found to be **800 M.m³**. **Ans.**

Table 18.16 *

Month	Inflow I M m ³	Outflow (O) M m ³	(I - O)		$\Sigma (I - O)$	
			+ve values	-ve values	Cumulative excess (M m ³)	Cumulative deficit (M m ³)
			Excess (M m ³)	Deficit (M m ³)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
June	250	150	100		100	
July	350	150	200		300	
Aug	400	200	200		500	
Sept	200	250		50		50
Oct	150	350		200		250
Nov	150	400		250		500
Dec	100	250		150		650
Jan	50	200		150		800*
Feb	150	150	0		0	(Peak value in col 6 & 7)
March	300	150	150		150	
April	400	100	300		450	
May	450	250	200		650	
June	150	350		200		200
July	200	300		100		300
Aug	450	100	350		350	

* Max. of all the values in col (6) & (7)

Example 18.9. Solve Example 18.7 analytically without using mass curve.
Solution. The data given in example 18.7 is used in Table 18.17 to compute ($I - O$) values in col (4) and (5) [$-ve$ i.e. deficit values in col (4), and $+ve$ i.e. Excess values in col (5)]. The values of $\Sigma (I - O)$ are finally computed in col (6) & (7). as shown :

Table 18.17

Year	Inflow (I) Mm ³	Outflow (O) Mm ³	I - O		$\Sigma (I - O)$	
			- ve	+ ve	Cumulative Deficit Mm ³	Cumulative Surplus Mm ³
			Deficit Mm ³	Surplus Mm ³		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1962	1642	2079	437		437	
63	2404	2079		325		325
64	2819	2079		740		1065*
65	1575	2079	504		504	
66	1596	2079	483		987	
67	2553	2079		474		474
68	1842	2079	237		237	
69	2199	2079		120		120

Out of all the values of col. (6) and (7), the max. value is 1065 Mm³, which represents the min. storage required to accommodate this 1065 Mm³ surplus water entering during the years 1963 and 1964. **Ans.**

Gross storage reqd. = Dead storage + Live storage
= 20% of Live storage + Live storage
= 1.2 Live storage = $1.2 \times 1065 \text{ Mm}^3 = 1278 \text{ Mm}^3$ **Ans.**

Example 18.10. The yield of water in Mm³ from a catchment area during each successive month is given in the table below :

1.4	2.1	2.8	8.4	11.9	11.9
7.7	2.8	2.52	2.24	1.96	1.68

Determine the minimum capacity of a reservoir required to allow the above volume of water to be drawn off at a uniform rate assuming that there is no loss of water over the spillway.

Solution. The total inflow of water in 12 months
= Summation of inflow values = 57.4 Mm³.
 \therefore Average monthly rate at which water is withdrawn to use the inflow fully = Av. demand rate = $57.4/12 = 4.78 \text{ Mm}^3$.
Now, to draw the mass curve of inflow, the cumulative inflow values are worked out in table 18.18.

Table 18.18

Month	Yield (Mm ³)	Cumulative yield (Mm ³)
1	1.4	1.4
2	2.1	3.5
3	2.8	6.3
4	8.4	14.7
5	11.9	26.6
6	11.9	38.5
7	7.7	46.2
8	2.8	49.0
9	2.52	51.52
10	2.24	53.76
11	1.96	55.72
12	1.68	57.40

The mass inflow curve is now plotted, as shown in Fig. 18.17. A line parallel to the demand rate line is now drawn through the high point A_1 on the inflow mass curve, as shown. The maximum departure between the inflow mass curve and this line, i.e. B_1C_1 , gives the min. storage capacity reqd. This value is read out as 20.78 Mm^3 . **Ans.**

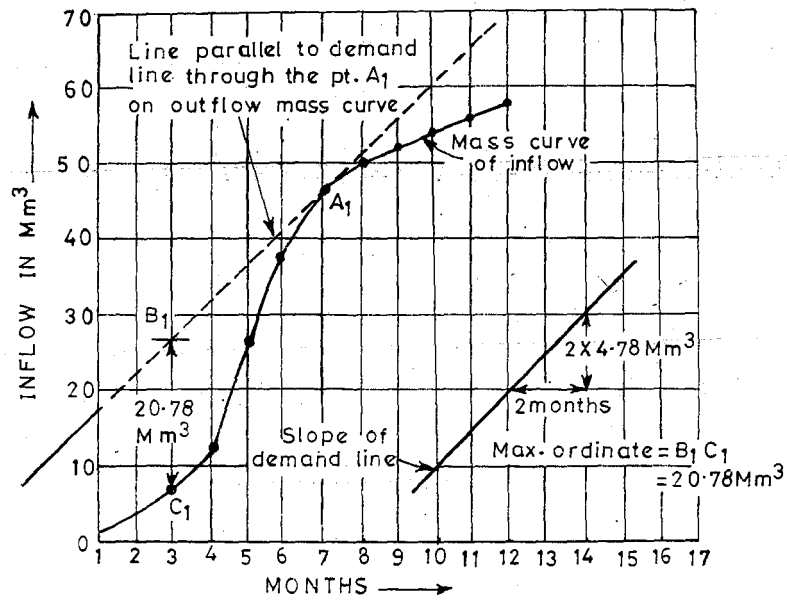


Fig. 18.17.

Analytically, the solution to the problem can be worked out, as shown in table 18.19.

Table 18.19

Month	Inflow Mm^3	Outflow (Demand) Mm^3	Deficit Mm^3	Surplus Mm^3	Cumulative Deficit Mm^3	Cumulative Surplus Mm^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1.4	4.78	3.38			
2	2.1	4.78	2.68			
3	2.8	4.78	1.98		8.04	
4	8.4	4.78		3.62		
5	11.9	4.78		7.12		
6	11.9	4.78		7.12		
7	7.7	4.78		2.92		20.78
8	2.8	4.78	1.98			
9	2.52	4.78	2.26			
10	2.24	4.78	2.54			
11	1.96	4.78	2.82			
12	1.68	4.78	3.10		12.70	

The highest value in col. (6) and 7 (as no spilling is allowed), is 20.78 Mm^3 , which gives the min. required reservoir storage. **Ans.**

Example 18.11. *The runoff data for a river during a lean year are given below :*

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
River flow in 10 ⁶ m ³	140	27	35	26	16	48	212	180	116	92	67	37

What is the maximum uniform demand that can be met? What is the storage capacity required to meet this demand? What minimum initial storage is necessary? When does the reservoir become empty?

Solution. Total inflow in the year = Summation of the given monthly discharges
= 996 M. m³.

Average monthly rate at which water can be withdrawn to avoid any wastage i.e. max average monthly rate

$$= \frac{996}{12} = 83 \text{ Mm}^3 \text{ Ans.}$$

Now, to determine storage capacity etc. we carry out the computations in table 18.20.

Table 18.20

Month end	Monthly Inflow in reservoir in Mm ³ .	Monthly outflow from reservoir in Mm ³ .	Monthly deficit in reservoir (to be supplied from storage) in Mm ³	Monthly surplus in reservoir in Mm ³	Consecutive cumulative deficit in Mm ³	Consecutive cumulative surplus in Mm ³	Net water available in the reservoir in Mm ³
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Jan	140	83		57		57	(+) 57
Feb	27	83	56				
March	35	83	48				
April	26	83	57				
May	16	83	67				
June	48	83	35		263		(-) 206
July	212	83		129			
Aug	180	83		97			
Sept	116	83		33			
Oct	92	83		9		268	(+) 62
Nov	67	83	16				
Dec	37	83	46		62		0

The highest value in col. (6) and (7) is 268 Mm³, which represents the minimum storage capacity required to meet the demand without any spilling. **Ans.**

To compute the min. initial storage, we compute in col. (8), the net storage left in the reservoir with the above inflows and outflows. The maximum negative storage here works out to be 206 Mm^3 . In order that the reservoir fully meets the demand with the above inflows & outflows, there should be no negative storage in it, and in the limiting case the max. negative storage should be equal to zero. Hence, the min. initial storage in the reservoir should be 206 Mm^3 , which will just meet the shortage created in the reservoir by *June end*, when the reservoir will become empty. **Ans.**

Example 18.12. *The storage capacity of a reservoir for a flood control project is to be determined. The estimated cost of damage if the emergency spillway is topped is Rs. 10 lacs for each event. The interest rate is 6% and the reservoir life is 50 years. Six reservoir designs of different storage capacities ; the probabilities of exceeding those capacities in any given year, and the estimated initial cost for construction of the reservoir are as follows :*

Reservoir design number	1	2	3	4	5	6
Flood storage capacity in lac cubic metres	30	35	40	45	50	55
Probability of exceeding storage capacity in any given year	0.15	0.10	0.06	0.04	0.02	0.01
Estimated initial cost, in lac rupees	25	30	35	40	45	50

Determine the optimal storage capacity of the reservoir.

(A.M.I.E. 1989)

Solution. The required computations are carried out in table 18.21 below.

Table 18.21

Reservoir Design No.	Reservoir Capacity in lakh cum	Probability of exceeding storage capacity	No. of times the flood is likely to exceed the capacity in 50 yrs.	Damages caused due to flood exceeding capacity in Rs. Lakh = Rs. 10 lakh \times Col (4)	Extra benefit over proposal No. 1 of lowest storage as datum; Rs. 75 lakh - col. (5)	Initial cost of each proposal in Rs. Lakh	Extra costs involved over proposal 1 as datum, in Rs. Lakh Col. 7 - Rs. 25 lakh	Net additional benefit in Rs. lakh, col. (6) - col. (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	30	15%	7.5	75	—	25	—	—
2	35	10%	5	50	25	30	5	20
3	40	6%	3	30	45	35	10	35
4	45	4%	2	20	55	40	15	40
5	50	2%	1	10	65	45	20	45
6	55	1%	0.5	5	70	50	25	45

The above table computes the damages caused by overflowing for each proposal, the max. damage of Rs. 75 lakhs being for proposal 1 - having min. reservoir capacity; and the min. damage of Rs. 5 lakh being for proposal 6 - having max. storage capacity.

The additional benefits thus provided by proposals 2 to 6 over proposal 1 are computed in col (6) as 75 lakhs – col (5). Extra costs are also worked out in col (8).

These calculations reflect that in proposal 2, we spend extra Rs. 5 lakh (over proposal 1) and get a benefit of 25 lakhs (over proposal 1 of course) ; i.e. a net benefit of Rs. 20 lakh. Similarly, in proposal 3, we spend extra Rs. 10 lakhs & get a benefit of Rs. 45 lakhs over proposal 1; i.e. a net benefit of Rs. 35 lakhs. Similarly, we find that our net benefit increases up to proposal 5, and afterwards, as in proposal 6, it becomes equal to that in proposal 5. Hence, the increase in reservoir capacity up to proposal 5 gives us increasing net returns, but thereafter we don't get any extra net returns. Investing more, beyond proposal 5, would, therefore, not be optimal, and thus, proposal 5 can be considered as the most optimum design.

Hence, the optimum storage capacity of the reservoir will correspond to that of proposal 5, i.e. 50 lakh m^3 . Ans.

18.10. Estimation of Demands and Optimal Reservoir Operations

In the previous articles, it was explained as to how the reservoir capacity can be determined for satisfying a certain given downstream demand or *vice versa*. This demand may be constant or may vary throughout the year.

The demand pattern mainly depends upon the purpose for which the reservoir has been constructed. Hence, it has to be different for different types of reservoirs. Moreover, the demands are man made, and should be adjusted in such a way that the maximum benefits can be obtained with the minimum cost. If by reducing the demands slightly, a lot of expenditure can be saved, then we must go in for that. This is known as *optimal planning* for dam reservoirs, and involves economic considerations, discussed afterwards.

Demand patterns for various types of reservoirs are explained below :

(i) *Single Purpose Conservation Reservoir*. A single purpose conservation reservoir shall store water only to meet one particular demand, such as water supply or irrigation or hydroelectric power generation, etc. In all such cases, the demand for water can be predicted with a fair degree of accuracy for different parts of the year, and the storage capacity can be worked out, as explained earlier. The storage capacity so worked out should be otherwise feasible and should be justified on cost benefit considerations.

(ii) *Single Purpose Flood Control Reservoir*. In case of a single purpose flood control reservoir, there is no specific downstream demand at all, except to satisfy that the downstream release should not exceed the safe carrying capacity of the channel. Since the effect of constructing a flood control reservoir is to moderate and reduce the flood peaks by absorbing certain volume of flood, and then gradually releasing it when the flood subsides (Fig. 18.18), the capacity of such a reservoir should be sufficient to absorb this excess volume of flood.

It, therefore, becomes evident that the capacity of such a

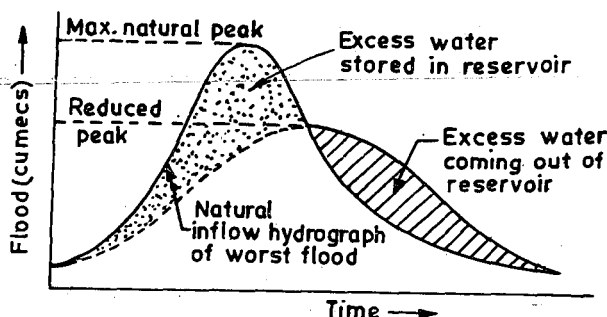


Fig. 18.18.

reservoir does not depend upon the pattern of demand but mainly depends upon the hydrograph of the worst flood that is likely to enter this reservoir and also upon the downstream permissible H.F.L. and the safe carrying capacity of the channel.

The hydrograph of the worst inflow flood can be found from the hydrological investigations, and the safe peak rate of outflow can be determined from the downstream channel conditions. The capacity of the reservoir required to moderate the inflow peak to a value equal to or less than the downstream safe peak can then be found by hit and trial method with the help of *flood routing*. Flood routing is the process by which the hydrograph of the moderated flood can be determined, and is explained a little later.

(iii) *Multipurpose Reservoirs*. Single purpose reservoirs are seldom constructed these days. Reservoirs are therefore, generally designed to serve more than one purpose. For example, a reservoir constructed for conserving water for irrigation may be combined with its flood control purpose. The head available may simultaneously be utilised for generating hydro-electric power. This combination of irrigation, flood control, and power is generally adopted in designing multipurpose reservoirs in India.

For proper optimal planning of such a multipurpose reservoir, a schedule of operations must be finalised from a number of tentative schedules drawn on the basis of available data and past similar experiences. The schedule which gives maximum total benefits for the various design purposes, without encroaching upon the lower or upper limits of storage, is accepted for estimation and for initial operations. This schedule may be tried and further modified on the basis of the past experience and future needs.

A multipurpose reservoir constructed on a perennial snow fed river in North India (such as Bhakra reservoir) can be operated on the following lines :

Fig. 18.19 shows the typical schedule of operations for such a reservoir. The reservoir water will normally fluctuate between minimum pool level and normal pool level (i.e. maximum conservation level) for satisfying irrigation needs. The minimum pool level will ensure the generation of firm power, as the water level shall not normally

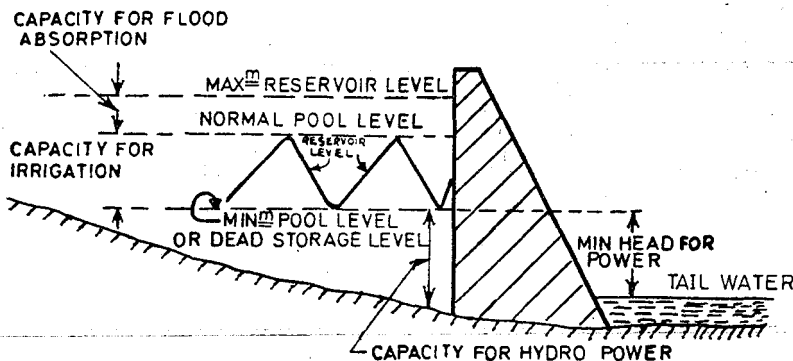


Fig. 18.19.

be allowed to go below the minimum pool level. Any serious flood which may occur after the reservoir is full, shall be absorbed between the normal pool level and the maximum pool level, at all times. As soon as the water level reaches the normal pool level, the spillway will start functioning. But the rate of water discharged over the spillway shall be in accordance with the downstream safe carrying capacity of the

channel, and hence, may be less than the rate of inflow. The water shall, then, be stored temporarily between the normal pool level and the maximum reservoir level, and discharged at a safe rate. As the flood subsides, and the excess water goes out slowly, the water level will again fall up to the normal pool level, and the reservoir will be ready to absorb another flood.

For irrigation, we generally require water from April to June for Kharif crops, and from middle of November to middle of February for Rabi crops. There are heavy rains during monsoon period of say June to middle of October. At the start of the monsoon season, the reservoir is quite depleted because of heavy outflow for Kharif crops and constant release for power generation. Thus, just before the monsoon, a large reservoir capacity is available, which not only conserves the water but also serves to control floods that may occur at this time. During the monsoon, the irrigation demand will almost be nil and water shall be released only for power generation. The reservoir level will, therefore, go on increasing steadily and will be allowed to go up to the normal pool level by the end of normal monsoon season, say middle of October. Any untimely flood occurring after the reservoir is full up to normal pool level, shall be absorbed between normal pool level and the maximum reservoir level, as explained earlier.

After the monsoon is over, the inflow is considerably reduced (almost nil) and water is constantly drawn for Rabi crops and power generation. Due to this, the reservoir level goes on falling up to about middle of February, when it is usually the lowest, somewhere near the minimum pool level (*i.e.* dead storage level). At this time, snow starts melting in the head reaches of the river, which augments the river discharge. Some winter cyclonic rains during December and January may also be helpful in augmenting the reservoir storage. From February-March onwards, there are large supplies due to melting of snow, and an equal burden is to be absorbed by the reservoir for satisfying the Kharif irrigation withdrawals. Thus, during April, May and June, the reservoir level may rise or fall slightly at slow rate; and at the start of monsoon, *i.e.* at the end of June or beginning of July, the reservoir will be quite depleted and prepared for full replenishment of its supplies.

FLOOD ROUTING OR FLOOD ABSORPTION

The hydrograph of a flood entering a reservoir, will change in shape as it emerges out of the reservoir, because certain volume of its water is stored in the reservoir temporarily and is let off as the flood subsides. The base of the hydrograph, therefore, gets broadened, its peak gets reduced, and, of course, the time of peak is delayed. The extent by which the inflow hydrograph gets modified due to the reservoir storage can be computed by a process known as *flood routing*, and more particularly as *reservoir routing* (to differentiate it from routing through 'river channels').

Since the flood protection reservoirs are generally located many km upstream of the cities which are to be saved against floods, it is sometimes necessary to route the outflow hydrograph of the reservoirs up to these downstream localities. The reservoir outflow hydrograph may then be routed through this much length of river channel, so as to obtain the final shape of the hydrograph at the affected cities. This routing, in which the stream itself acts like an elongated reservoir, is known as *channel routing* or *river routing*.

A variety of routing methods are available, and they can be broadly classified into two categories, viz

(i) *hydrologic routing* ; and

(ii) *hydraulic routing*

Hydrologic routing methods employ essentially the *equation of continuity* ; whereas the *hydraulic routing methods* employ the *continuity equation together with the equation of motion of unsteady flow*. The basic differential equations used in hydraulic routing are popularly known as *St. Venant equations*, and afford a better description of the unsteady flow than the hydrologic methods. We shall, however, confine ourselves in the book to the hydrologic methods only.

18.11. Hydrologic Reservoir Routing Methods

The passage of a flood wave through a reservoir or a river reach is an *unsteady flow* phenomenon. In hydraulics, we classify it as a *gradually varied flow*. The *equation of continuity* used in all the hydrologic routing methods, as the primary equation, states that the difference between the inflow and the outflow rate is equal to the rate of change of storage ; i.e.

$$I - O = \frac{dS}{dt} \quad \dots(18.9)$$

where I = Inflow rate
 O = Outflow rate
 S = Storage

Alternatively, in a small time interval Δt , the difference between the total inflow volume and the total outflow volume is equal to the change in storage volume ; viz.

$$\underbrace{\bar{I} \Delta t}_{\substack{\text{(Inflow volume)} \\ I(v)}} - \underbrace{\bar{O} \Delta t}_{\substack{\text{(Outflow vol.)} \\ O(v)}} = \Delta S \quad \dots(18.10)$$

where \bar{I} = Average inflow (rate) in time Δt
 \bar{O} = Average outflow (rate) in time Δt
 ΔS = change in storage during the time Δt

Since $\bar{I} = \frac{I_1 + I_2}{2}$;
 $\bar{O} = \frac{O_1 + O_2}{2}$;
 $\Delta S = S_2 - S_1$;

where suffixes 1 and 2 denote the beginning and the end of the time interval Δt .

Eq. (18.10) can then be written as :

$$\left(\frac{I_1 + I_2}{2} \right) \Delta t - \left(\frac{O_1 + O_2}{2} \right) \Delta t = S_2 - S_1 \quad \dots(18.11)$$

The time interval Δt should be sufficiently short, so that the inflow and outflow hydrographs can be assumed to be in straight line, in that time interval. Moreover, Δt must be shorter than the time of transit of the flood wave through the reservoir or the given river reach.

The above relationship seems to be very simple, but its evaluation is not easily possible without drastic simplifying assumptions. This is because of the fact that the relations between time and rate of inflow, elevation and storage of reservoir, and

elevation and rate of outflow, cannot be expressed by simple algebraic equations. They are, respectively represented by the *inflow flood hydrograph*, the *elevation-storage curve*, and the *outflow-elevation curve*. The first two curves obviously cannot be represented by any simple equations, and the third may be represented by the spillway-discharge equation ($Q = 1.71 LH^{3/2}$) only if, the discharge through the outlets is neglected. If the discharge through the outlets is also not neglected, then all the three curves will be unamenable to simple mathematical treatment, without drastic assumptions.

Several procedures have, however, been suggested by different investigators to solve the above basic equation (Eq. 18.11) by rearranging the components in different manners. Depending upon the different procedures adopted for solving the above basic equation, the following hydrologic methods may be used for reservoir routing :

- (1) *Trial and Error method*
- (2) *Modified Pul's method or Storage indication method ; and*
- (3) *Goodrich method.*

The first method is discussed here, while the detailed description of other methods is available in authors another book titled "*Hydrology and Water Resources Engineering*" and may be referred to in specific needs.

18.11.1. Trial and Error Method of Reservoir Routing. Trial and Error method is widely adopted with the assistance of computers to reduce the time taken in long calculations involved in this method. This method arranges the basic routing equation (Eq. 18.11), as follows :

$$\frac{I_1 + I_2}{2} \cdot \Delta t = \frac{O_1 + O_2}{2} \cdot \Delta t + (S_2 - S_1) \quad \dots(18.11 a)$$

The procedure involves assuming of a particular level in the reservoir at the end of the interval Δt , and computing the values on the right side of the above Eq. (18.11 a).

The summation of $\frac{O_1 + O_2}{2} \cdot \Delta t$ and $(S_2 - S_1)$ is then compared with the known value of

$\frac{I_1 + I_2}{2} \cdot \Delta t$. If the two values tally, then the assumed reservoir elevation at the end of the interval is supposed to be O.K. ; otherwise this is changed, and the process is repeated till the required matching is obtained.

This method gives quite reliable results, provided the chosen time interval (Δt) is sufficiently small, so that the mean of the outflow rates at the start and the end of the given interval may be taken as the average throughout the interval.

Procedure : The following detailed procedure may be adopted in this method, to complete the involved computations :

- Data to be given : (i) the inflow hydrograph
 (ii) Elevation capacity curve or Elevation area curve
 (iii) Elevation outflow curve.

Steps involved in computations

(i) Divide the inflow flood hydrograph into a number of small intervals. The time interval should be so chosen, as not to miss the peak values.

(ii) Fix the normal pool level at which the spillway crest is provided, and the level at which the flood enters the reservoir ; the two are generally taken to be the same, as

it is assumed that this worst design flood enters the reservoir only after the reservoir is full up to the normal pool level.

(iii) Work out the spillway and the outlet discharge rating curves, if not given.

(iv) Work out the elevation-capacity curve for the reservoir from the elevation-area curve, if the former is not given, using cone formula, *i.e.* $V = \Sigma \frac{h}{3} [A_1 + A_2 + \sqrt{A_1 A_2}]$, where h is the contour interval. (18.12)

(v) Start with the first interval and compute the total inflow during the interval by multiplying the average inflow rate at the beginning and the end of the interval, with the period of the interval.

$$\therefore I_{(v)} = \frac{I_1 + I_2}{2} (\Delta t)$$

where I_1 = Inflow rate at the start of the interval

I_2 = Inflow rate at the end of the interval

Δt = Duration of the interval

$I_{(v)}$ = Total inflow volume during the interval.

(vi) The reservoir level at the start of the flood (*i.e.* start of first interval) is known. Assume a trial value for the reservoir level at the end of the interval.

(vii) compute the total outflow during the interval

$$O_{(v)} = \frac{O_1 + O_2}{2} \cdot \Delta t$$

where O_1 = Outflow rate at the start of the interval, corresponding to the given reservoir level.

O_2 = Outflow rate at the end of the interval, corresponding to the assumed reservoir level.

Δt = Duration of the interval

$O_{(v)}$ = Total outflow volume during the interval.

(viii) Using the elevation-storage curve for the reservoir, determine the storage S_1 and S_2 at the beginning and the end of the interval, corresponding to the known and the assumed reservoir levels, respectively. Their difference $S_2 - S_1 = \Delta S$, represent the amount of flood stored in the reservoir during the interval.

(ix) Add the volume of outflow $O_{(v)}$ obtained in step (vii) to the values of ΔS obtained in step (viii), and compare it with the inflow volume $I_{(v)}$, calculated in step (v). The two values must be equal (*i.e.* $I_{(v)} = O_{(v)} + \Delta S$). If this is so, the assumed reservoir level is correct, otherwise, change it and repeat the procedure till this coincidence is obtained.

(x) All the above steps should be repeated for other time intervals, till the entire flood is routed or still further, till the reservoir level returns to pre-flood pool level.

(xi) Outflow ordinates are plotted so as to obtain the outflow hydrograph. The point at which it crosses the inflow hydrograph gives the peak outflow rate. From this time, the rate of outflow begins to fall due to decrease in the inflow rate.

(xii) The time lag between the two peaks is evaluated as to give the time lag.

An example has been solved to make the procedure very clear.

Example 18.13. *The inflow flood discharges for a possible worst flood are tabulated in Table 18.22 at suitable intervals starting from 0.00 hours on august 20, 1975.*

Table 18.22

Time from start in hr	0	6	12	18	24	30	36	42	48	51	60	66	78	90	102	114
Dis-charge in cumecs	0	50	280	610	1290	1900	2130	1900	1600	1440	1060	780	500	370	220	130

This flood approaches a reservoir with an uncontrolled spillway, the crest of which is kept at RL 140.0 m. Determine the maximum reservoir level and the hydrograph of the routed flood. Values of reservoir capacity (above spillway crest) and outflow discharge at various elevations are tabulated in Tables 18.23 and 18.24 respectively.

Table 18.23

Elevation in metres	140.0	141.0	142.0	143.0	144.0	145.0	146.0
Reservoir storage with spillway crest as datum in million cubic metres (m.c.m.)	0.0	15.0	35.0	60.0	95.0	140.0	240.0

Table 18.24

Elevation	Outflow discharge in cumecs
140.0	0
141.0	170
142.0	482
143.0	883
144.0	1,360
145.0	1,905
146.0	2,500

Solution. The elevation storage curve and the elevation outflow curve are plotted with the help of Tables 18.23 and 18.24, as shown in Fig. 18.20 and 18.21 respectively. The hydrograph of the given flood is plotted in Fig. 18.22. Flood routing is carried out by hit and trial method as shown in Table 18.25 and as explained earlier. This table is otherwise self-explanatory.

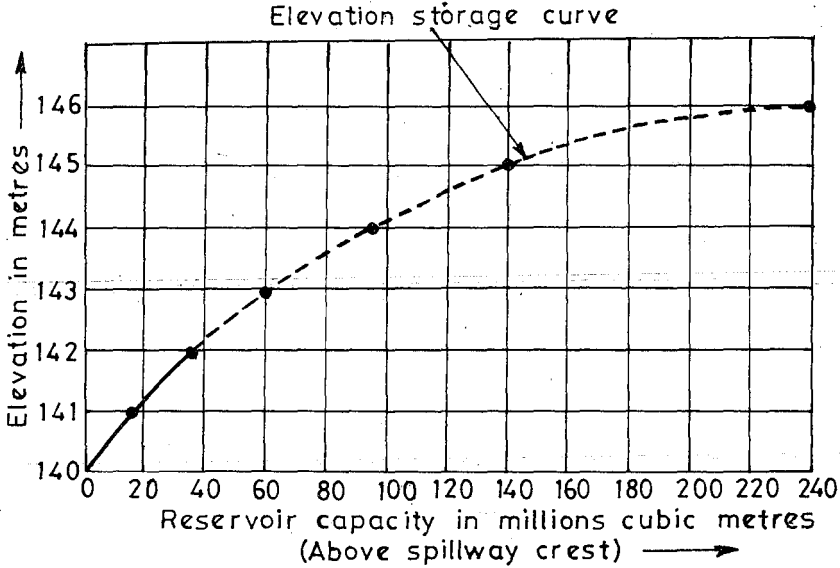


Fig. 18.20. Elevation Storage curve for example 18.13.

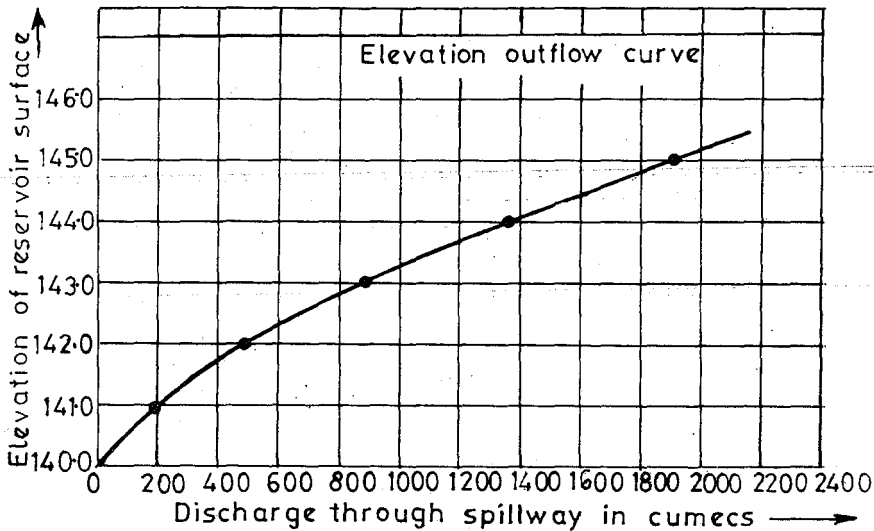


Fig. 18.21. Elevation-Outflow curve for example 18.13.

The outflow hydrograph is plotted from col. (7) of table 18.25, as shown in Fig. 18.22 by the dotted curve. The point at which it intersects the inflow hydrograph, represents the peak of the routed flood.

The peak of outflow flood works out to be 1458 cumecs, while the peak of the inflow flood was 2,130 cumecs. The time lag is found to be 15 hours (from Fig. 18.22). The maximum reservoir level is found from Table 18.25, to be 144.18 metres. **Ans.**

Table 18.25

			(1)	Time from start	
			(2)	Time interval (Δt)	
			(3)	Reservoir elevation at the beginning of interval	
Hrs.			(4)	Inflow at the start of interval (I_1)	
6			(5)	Inflow at the end of interval (I_2)	
140.0			(6)	Volume of inflow during the interval $I(V) = (I_1 + I_2) / 2$	
0.0			(7)	Outflow at the start of interval (O_1)	
50			(8)	Trial reservoir elevation at the end of interval	
0.54			(9)	Outflow at the end of interval (O_2)	
0.0			(10)	Mean outflow volume $O(V) = [(O_1 + O_2) / 2] \Delta t$	
140.03			(11)	Storage capacity at start of interval (S_1)	
5.67			(12)	Storage capacity at the end of interval (S_2)	
0.88			(13)	Change in storage during the interval (ΔS) = ($S_2 - S_1$)	
0.01			(14)	Outflow volume + change in storage; i.e. $O(V) + \Delta S$	
0.06			(15)	Compare $O(V) + \Delta S$ with $I(V)$, i.e. col. (14) with col. (6) and Remarks	
0.06					
0.01					
0.0					
1.2					
0.525					
1.2					
0.525					
1.26					
0.535					
Large and hence change the trial value O.K.					

...contd

Table 18.25 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
6-12	6	140.03	50	280	3.56	0.88	140.3	28.7	0.31	0.525	4.0	3.475	3.785	large
							140.25	21.2	0.23	"	3.85	3.325	3.555	O.K.
12-18	6	140.25	280	610	9.62	21.2	140.5	60.0	0.88	3.85	8.0	4.15	5.03	small
							141.0	170.0	2.06	"	15.0	11.15	13.21	large
							140.8	121.6	1.54	"	13.0	9.15	10.69	large
							140.75	110.2	1.42	"	12.05	8.20	9.62	O.K.
18-24	6	140.75	610	1290	20.5	110.2	141.5	312.2	4.57	12.05	26.0	13.95	18.52	small
							141.6	343.0	4.89	"	27.7	15.65	20.54	O.K.
24-30	6	141.06	1290	1900	34.4	343.0	142.5	672	10.95	27.7	47.0	19.3	30.25	small
							142.6	713	11.4	"	50.0	22.3	33.7	small
							142.63	725	11.52	"	50.5	22.8	34.32	O.K.
30-36	6	142.63	1900	2130	43.6	725	143.5	1113	19.85	50.5	76.0	25.5	45.35	large
							143.4	1065	19.23	"	73.0	22.5	41.73	small
							143.45	1087	19.55	"	74.5	24.0	43.55	O.K.
36-42	6	143.45	2130	1900	43.6	1087	144.0	1360	26.44	74.5	95.0	20.5	46.94	large
							143.95	1335	26.16	"	92.0	17.5	43.65	O.K.
42-48	6	143.95	1900	1600	37.8	1335	144.5	1620	13.9	92.0	115.0	23.0	64.9	large
							144.2	1461	30.2	"	102.0	10.0	40.2	large
							144.15	1435	29.9	"	100.0	8.0	37.9	O.K.

...contd

Table 18.25 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
48-51	3	144.15	1600	1440	16.4	1435	114.18	1458*	15.5	100.0	101.0	1.0	16.5	O.K.
51-60	9	144.18*	1440	1060	40.5	1458*	144.10	1410	46.5	101.0	98.0	-3.0	43.5	large
							144.05	1385	46.1	"	95.5	-5.0	41.1	O.K.
60-66	6	144.05	1060	780	19.9	1385	143.8	1258	28.5	95.5	87.0	-8.5	20.0	O.K.
66-78	12	143.80	780	500	27.8	1258	142.2	974	48.2	87.0	67.0	-20.0	28.2	O.K.
78-90	12	143.20	500	370	18.8	974	142.60	713	35.4	67.0	50.0	-17.0	18.4	O.K.
90-102	12	142.60	370	220	12.7	713	142.0	482	25.8	50.0	35.0	-15.0	10.8	small
							142.08	510	26.5		36.2	-13.8	12.7	O.K.

*Results : Peak rate of outflow = 1,458 cumecs
Maximum reservoir level = 144.18 metres
Time lag = 15 hours

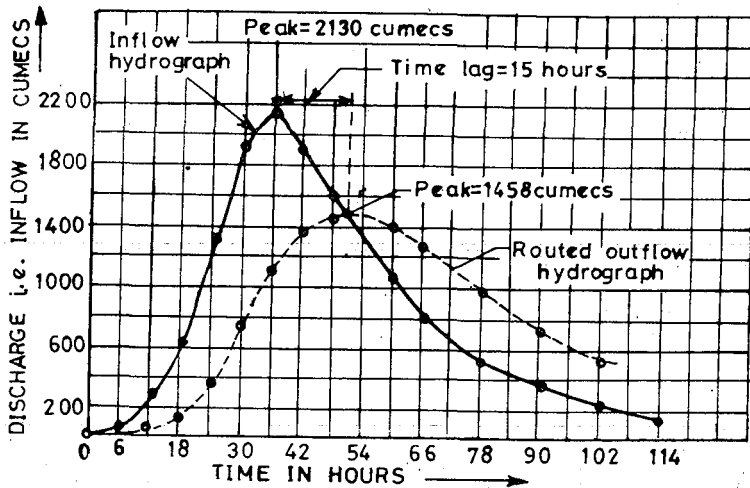


Fig. 18.22. Hydrograph of the given flood and that of the routed flood for example 18.10.

Example 18.14. The hydrograph of inflow to a reservoir is given in the table below.

Time(days)	0	2	4	6	8	10	12	14	16	18	20
Flow (m ³ /s)	60	115	425	550	440	320	260	200	150	110	60

The reservoir is full at the start of the flood inflow. The storage S of reservoir above the spillway crest, is given in million cubic metres by : $S = 8.64 h$, where h is the head in metres above the crest. The discharge over the spillway is given in cumecs by $Q = 60 h$. Find the head over the spillway crest at the end of 8th day of the flood.

Solution. Let h_1 be the head over the spillway crest at the end of 2 days ; h_2 at the end of 4 days ; h_3 at the end of 6 days ; and h_4 at the end of 8 days. Now we will consider the position, interval by interval.

(i) 0—2 day interval

Inflow $I_1 = 60$ cumecs.

$I_2 = 115$ cumecs.

$$\therefore I_{(v)} = \left(\frac{I_1 + I_2}{2} \right) t = \left(\frac{60 + 115}{2} \right) 2 \times (24 \times 60 \times 60) \text{ m}^3 = 15.12 \text{ M.m}^3.$$

Outflow $O_1 = 0$

$O_2 = 60 h_1$

$$O_{(v)} = \frac{O_1 + O_2}{2} \cdot t = \left(\frac{60 h_1}{2} \right) \times 2 \times (24 \times 60 \times 60) \text{ m}^3 = 5.184 h_1 \text{ M.m}^3.$$

Storage $S_1 = 0$

$S_2 = 8.64 h_1 \text{ M.m}^3$

$$\therefore \Delta S = 8.64 h_1$$

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RESERVOIRS AND PLANNING FOR DAM RESERVOIRS

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Now using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$15.12 = 5.184 h_1 + 8.64 h_1 \quad \text{or} \quad 15.12 = 13.824 h_1$$

$$\therefore h_1 = \frac{15.12}{13.824} = 1.093 \text{ m.}$$

(ii) 2—4 day interval

Inflow $I_1 = 115$ cumecs ; $I_2 = 425$ cumecs

$$I_{(V)} = \left(\frac{115 + 425}{2} \right) 2 \times 86400 \text{ m}^3 = 46.656 \text{ Mm}^3.$$

Outflow $O_1 = 60 h_1$; $O_2 = 60 h_2$

$$O_{(V)} = \left(\frac{60h_1 + 60h_2}{2} \right) 2 \times (86400) \text{ m}^3 = (h_1 + h_2) \times 5.184 \text{ Mm}^3.$$

Storage $S_1 = 8.64 h_1$; $S_2 = 8.64 h_2$

$$\Delta S = 8.64 (h_2 - h_1)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$46.656 = (h_1 + h_2) 5.184 + 8.64 (h_2 - h_1)$$

Using $h_1 = 1.093 \text{ m}$, we get

$$46.656 = 5.666 + 5.184 h_2 + 8.64 h_2 - 9.443$$

$$\text{or} \quad 50.433 = 13.824 h_2 \quad \text{or} \quad h_2 = 3.648 \text{ m.}$$

(iii) 4—6 day interval

Inflow $I_1 = 425$ cumecs ; $I_2 = 550$ cumecs

$$\therefore I_{(V)} = \frac{425 + 550}{2} \times 2 \times 86400 \text{ m}^3 = 84.24 \text{ M.m}^3.$$

Outflow $O_1 = 60h_2$; $O_2 = 60h_3$

$$O_{(V)} = \frac{60h_2 + 60h_3}{2} \times 2 \times 86400 \text{ m}^3 = (h_2 + h_3) 5.184 \text{ M.m}^3.$$

Storage $S_1 = 8.64h_2$; $S_2 = 8.64h_3$

$$\Delta S = 8.64 (h_3 - h_2)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$84.24 = (h_2 + h_3) 5.184 + 8.64 (h_3 - h_2)$$

Using $h_2 = 3.648 \text{ m}$ in this eqn., we get

$$84.24 = 18.911 + 5.184 h_3 + 8.64 h_3 - 31.518$$

$$\therefore 96.848 = 13.824 h_3 \quad \therefore h_3 = 7.00 \text{ m.}$$

(iv) 6—8 day interval.

Inflow $I_1 = 550$ cumecs ; $I_2 = 440$ cumecs

$$I_{(V)} = \left(\frac{550 + 440}{2} \right) 2 (86400) \text{ M.m}^3 = 85.536 \text{ M.m}^3.$$

Outflow $O_1 = 60 h_3$; $O_2 = 60 h_4$

$$O_{(V)} = \frac{60h_3 + 60h_4}{2} \times 2 \times 86400 \text{ m}^3 = (h_3 + h_4) 5.184 \text{ M.m}^3.$$

Storage $S_1 = 8.64 h_3$; $S_2 = 8.64 h_4$

$$\therefore \Delta S = S_2 - S_1 = 8.64 (h_4 - h_3)$$

Using $h_3 = 7.00$ m in this eqn., we get

$$85.536 = 36.288 + 5.184 h_4 + 8.64 h_4 - 60.48$$

or $109.728 = 13.824 h_4$ or $h_4 = 7.94$ m.

Hence, the head over the spillway crest at the end of the 8th day = 7.94 m. **Ans.**

Example 18.15. A small reservoir has an area of 5000 hectares ($50 \times 10^6 \text{ m}^2$) at spillway crest level. Banks are essentially vertical above the spillway crest level. The spillway is 50 m long and has a coefficient C of 2.2 in equation $q = CH^{3/2}$. The inflow to the reservoir is given in the table below :

Time from start (hrs.)	0	6	12	24	30	36	42	48	54
Inflow (m^3/s)	40	340	722	320	192	118	80	56	40

Compute the maximum outflow discharge over the spillway; and the reservoir level to be expected if the reservoir level was at the spillway crest at the start.

(Bhopal University, 1980)

Solution. As done in example 18.11, let us assume that the head over the spillway crest at the end of the various intervals be h_1, h_2, h_3, h_4 metres. Then

For interval 0—6 hr.

Inflow $I_1 = 40$ cumecs ; $I_2 = 340$ cumecs

$$I_{(V)} = \left(\frac{40 + 340}{2} \right) (6 \times 60 \times 60) \text{ m}^3 = 4.104 \text{ M.m}^3.$$

Outflow $O_1 = 0$

$$O_2 = 2.2 \times 50 \times h_1^{3/2} = 110 h_1^{3/2} \quad [\because Q_{\text{overspillway}} = C.L.H.^{3/2}]$$

$$O_{(V)} = \left(\frac{O_1 + O_2}{2} \right) \cdot t = \left(\frac{0 + 110 h_1^{3/2}}{2} \right) (6 \times 60 \times 60) \text{ m}^3 = 1.19 h_1^{3/2} \text{ M.m}^3.$$

Storage $S_1 = 0$

$$S_2 = (50 \times 10^6) h_1 \text{ m}^3. \quad [\because \text{Storage above the spillway is rectangular} \\ = \text{Area of the reservoir} \times \text{ht. of water}]$$

$$\therefore \Delta S = 50 h_1 \text{ M.m}^3.$$

Now, using $I_{(V)} = O_{(V)} + \Delta S$, we have

$$4.104 = 1.19 h_1^{3/2} + 50 h_1 \quad \text{or} \quad h_1^{3/2} + 42 h_1 - 3.45 = 0$$

Solving by hit and trial, we get

$$h_1 = 0.082 \text{ m.}$$

For interval 6—12 hr.

$$I_{(V)} = \left(\frac{340 + 722}{2} \right) (6 \times 60 \times 60) \text{ m}^3 = 11.47 \text{ M.m}^3$$

$$O_{(V)} = \frac{110 h_1^{3/2} + 110 h_2^{3/2}}{2} \cdot (6 \times 60 \times 60) \text{ m}^3 = (h_1^{3/2} + h_2^{3/2}) 1.19 \text{ M.m}^3$$

$$\Delta S = 50 (h_2 - h_1) \text{ M.m}^3$$

$$\therefore 11.47 = 1.19 (h_1^{3/2} + h_2^{3/2}) + 50(h_2 - h_1)$$

Using $h_1 = 0.082$, we have

$$9.65 = 0.02 + h_2^{3/2} + 42h_2 - 3.44 \quad \text{or} \quad 13.07 = h_2^{3/2} + 42h_2,$$

$$\text{or} \quad h_2^{3/2} + 42h_2 - 13.07 = 0 \quad \therefore \quad h_2 = 0.31 \text{ m.}$$

For interval 12—24 hr.

$$\text{Inflow} \quad I_{(V)} = \left(\frac{722 + 320}{2} \right) (12 \times 60 \times 60) \text{ m}^3 = 22.51 \text{ M.m}^3$$

$$\text{Outflow} \quad O_1 = 110 h_2^{3/2}; \quad O_2 = 110 h_3^{3/2}$$

$$O_{(V)} = \frac{110 (h_2^{3/2} + h_3^{3/2})}{2} \times (12 \times 60 \times 60) \text{ m}^3 = 2.38 (h_2^{3/2} + h_3^{3/2}) \text{ M.m}^3$$

$$\text{Storage} \quad S_1 = 50 h_2 \quad S_2 = 50 h_3$$

$$\Delta S = 50 (h_3 - h_2)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$$\therefore \quad 22.51 = 2.38 (h_2^{3/2} + h_3^{3/2}) + 50 (h_3 - h_2)$$

Using $h_2 = 0.31$ m, we have

$$9.46 = 0.17 + h_3^{3/2} + 12.2 h_3 - 3.78$$

$$13.07 = h_3^{3/2} + 12.2 h_3.$$

$$\text{or} \quad h_3^{3/2} + 12.2 h_3 - 13.07 = 0 \quad \therefore \quad h_3 = 0.99 \text{ m.}$$

For interval 24—30 hr.

$$\text{Inflow} \quad I_{(V)} = \left(\frac{320 + 192}{2} \right) (6 \times 60 \times 60) \text{ m}^3 = 5.53 \text{ M.m}^3$$

$$\text{Outflow} \quad O_{(V)} = \frac{110 (h_3^{3/2} + h_4^{3/2})}{2} (6 \times 60 \times 60) = 1.19 (h_3^{3/2} + h_4^{3/2})$$

$$\Delta S = 50 (h_4 - h_3)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$$\therefore \quad 5.53 = 1.19 (h_3^{3/2} + h_4^{3/2}) + 50 (h_4 - h_3)$$

$$\text{or} \quad 4.65 = (h_3^{3/2} + h_4^{3/2}) + 9.04 (h_4 - h_3)$$

Using $h_3 = 0.99$, we have

$$4.65 = 0.99 + h_4^{3/2} + 9.04 h_4 - 8.95$$

$$\text{or} \quad h_4^{3/2} + 9.04 h_4 - 12.61 = 0 \quad \therefore \quad h_4 = 1.24 \text{ m.}$$

For interval 30—36 hr.

$$I_{(V)} = \left(\frac{192 + 118}{2} \right) (6 \times 60 \times 60) \text{ m}^3 = 3.35 \text{ M.m}^3.$$

$$O_{(V)} = 1.19 (h_4^{3/2} + h_5^{3/2})$$

$$\Delta S = 50 (h_5 - h_4)$$

$$\therefore \quad 3.35 = 1.19 (h_4^{3/2} + h_5^{3/2}) + 50 (h_5 - h_4)$$

$$\text{or} \quad 2.81 = (h_4^{3/2} + h_5^{3/2}) + 42(h_5 - h_4)$$

Using $h_4 = 1.24$ m, we get

$$2.81 = 1.38 + h_5^{3/2} + 42h_5 - 52.08$$

$$\text{or} \quad h_5^{3/2} + 42h_5 - 50.65 = 0 \quad \therefore \quad h_5 = 1.17 \text{ m}$$

Since $h_5 < h_4$, it means that the head has started reducing, reaching its max. value of **1.24 m**.

\therefore max. outflow discharge over the spillway

$$= 110 (1.24)^{3/2} \text{ cumecs.} = 151.8 \text{ cumecs.} \quad \text{Ans.}$$

and, the max. reservoir level will be 1.24 m higher than the spillway crest level, which cannot be worked out as the spillway crest level is not given. **Ans.**

RESERVOIR REGULATION

18.12. Rule Curves and Operating Tables for Reservoirs

Multipurpose reservoirs need to be operated and regulated efficiently with a high degree of intelligence, intuition, and experience, in order to ensure that *they are neither left partially empty by the end of the rainy season, nor they are found full at the time of arrival of a series of peak floods, leading to heavy releases, causing floods in the downstream valley.*

Reservoir regulation committees, consisting of experts are, therefore, generally constituted to ensure issue of proper and timely directives to the staff operating the gated openings of the reservoir, to avoid any bad and inefficient operations.

Guiding tables and curves, called **rule curves** or **guide curves**, are drawn in advance, and kept ready for use for the efficient regulation of the reservoir waters, with time. Such guiding curves are normally required only for the flood season, because for the rest of the year, the reservoir will only discharge water for irrigation and hydel needs. Sometimes, **reservoir regulation manuals** are also prepared and made available to the officers, dealing with the operation of the gates of the reservoir. Such manuals provide guidelines for gate operations, and for the overall maintenance and upkeep of the dam and the reservoir.

A rule curve, either in tabular or graphical form, as framed by the experts on the basis of the past data, broadly reflects the maximum reservoir levels to be achieved by the different dates of the rainy season. *Such a curve, therefore, reflects the vacant space to be left in the reservoir on different dates or weeks of the rainy season.* Such guides may have to be revised from time to time, based on their performance during their actual use.

A typical set of such rule curves (tabular form) being followed for the operation of *Hirakud dam reservoir** are shown in table 18.26 (a) and (b).

* Hirakud dam is constructed on Mahanadi river in Orissa State of India. It is 3 mile long composite dam, with max. height of 200 ft. Spillway capacity at FRL of 630' is 14.83 lakh cusecs. It intercepts a catchment of 32,200 sq. miles, and the catchment below the dam and up to the head of delta, i.e. Naraj, is about 18,000 sq. miles. There is no flood reserve or conservation reserve as such, and it is a general use reservoir. The river capacity at Naraj is about 9 lakh cusecs.

Table 18.26 (a) Rule curve for filling the Hirakud reservoir for conservation

<i>Date of the year</i>	<i>Permissible reservoir water level (Dead storage RL=590ft)</i>
11th August (on and upto)	600'
21st August	605'
1st September	617'
11th September	623'
21st September	627'
1st October (on and after)	630' (F.R.L.)

**Table 18.26 (b) Rule curve for flood releases
(under ordinary circumstances) for Hirakud dam reservoir**

<i>Range of levels RL in ft</i>	<i>Permissible outflow in lakh cusecs</i>	<i>Safe depletion rate in lakh cusecs</i>
600 — 610	2.5	4.0
610 — 615	3.0	
615 — 620	4.0	
620 — 625	4.5	
625 — 628	5.5	
628 — 630	6.5	

Prior to the above rules, Hirakud dam reservoir was following another set of rules, as given in table 18.27, which proved to be inadequate, leading to insufficient storage at the end of flood seasons for some of the years, and heavy discharges from the reservoir synchronising with floods in the lower catchment in some other years.

**Table 18.27. Old Rules, specified in the Manual for
Operation of Hirakud reservoir, which have now been
superceded by rules of tables 18.21 (a) and (b)**

<i>S. No. of Rule</i>	<i>Rule</i>
Rule 1	There should be no full impounding till 1st of September in normal years. The entire inflow should be let out ; care being exercised that the combined discharge of reservoir outflow and the runoff from the catchment below the dam does not exceed 10 lakh cusecs at Naraj. Generally, the inflow up to 5 lakh cusecs can be passed without hesitation. When the inflow is more than 5 lakh cusecs, then the hydrometeorological conditions in the lower catchment should be carefully studied and outflow determined.
Rule 2	Safety of dam should be the prime consideration on and at every occasion when floods are absorbed in the reservoir for purposes of moderating and regulating outflow in the reservoir. A level of + 625.0' should be deemed to be the safe level normally for this purpose.

Another typical rule curve (graphical form), as drawn and used for Maithon dam reservoir, under *D.V.C. System*^{*}, is shown in Fig. 18.23. This operation schedule shows that the reservoir, which will be quite depleted by the start of June, will be filled up fully up to *Monsoon storage level* (RL 480'), latest by 3rd week of September, and will be maintained full, till the end of November. The reservoir will be allowed to be depleted only from December onward.

Any flood, which occurs during the last week of September, October and November, will thus, find the reservoir full, and will have to be accommodated within the '*flood reserve*', i.e. between monsoon storage level (RL 480') and the max. permissible operational level of gates (RL 495').

Depending upon the intensity of the coming and likely floods after the reservoir is full, the outflows will be regulated and so manipulated that the discharge from the reservoir together with the discharge contributed by the downstream area, does not exceed the safe carrying capacity of the river farther downstream.

As and when a late flood enters the reservoir, the reservoir-gate-operators may adjust the outflow to a lower value in the beginning, but may have to increase the outflow, as the flood waves continue and the weather forecasts for continued rains are received. In this operation, a stage may reach when the outflow may become quite high, causing floods in the down valley.

While manipulating these outflows, chances of synchronisation of the discharge of the downstream uncontrolled catchment will also have to be considered properly.

Say for example, if we release 1 lakh cusecs discharge from the reservoir, and another 1 lakh cusecs gets added to it from the catchment of the downstream 1 km length of the channel, then eventually 2 lakh cusecs will be flowing in the channel farther down the 1 km length. This 2 lakh cusecs may prove quite harmful, as it may exceed the safe carrying capacity of the downstream river (below 1 km).

Another possibility for gate operators could then be to release lesser discharge, say 50,000 cusecs, which with 1 km downstream contribution of 1 lakh cusecs, will make up 1,50,000 cusecs, which may not prove dangerous to farther down. But releasing lesser quantity would cause the reservoir water level to rise rapidly, encroaching the '*flood reserve*' quickly, finally leading to a stage, wherein you may have to open the entire gates, passing say 4 lakh cusecs, to avoid over-topping and failure of the dam. This 4 lakh cusecs will eventually destroy the entire downstream area, and may prove worse than what would have happened, if 1 lakh cusecs would have been allowed in the beginning itself.

You can, thus, understand and appreciate as to how important it is, to properly manage the outflows from a reservoir. The intelligent and timely manipulations, accompanied by proper rain forecasts, may help us to avoid flooding of the downstream area.

^{*} D.V.C. (Damodar Valley Corporation) was constituted in India in July, 1948, on the line of T.V.A. (Tennessee Valley Authority) of USA. to plan and execute flood control works on river Damodar and its tributaries, which had created havoc in the year 1943, causing serious flood losses of the order of Rs. 8 crores, and cutting off the most important Calcutta city with the rest of the country for about ten weeks, due to breaches in G.T. Road and Rly link.

Consequently, 4 dams were constructed, including the Maithon on river Damodar, Konar on river Konar, and Panchat and Tallaiya on its tributary Barakar, along with a barrage at Durgapur (situated a few km downstream of the confluence of Damodar and Barakar. Konar and Tallaiya are smaller dams located in the upper reaches, whereas, Maithon and Panchat are bigger dams in lower reaches, bearing most of the flood burden.

Maithon is located about 13 km above the confluence, and 21 km northwest of Asansol and 40 km east of Dhanbad. Like Konar, it is partly earthen and partly of concrete. The primary purpose of the dam is flood control; whereas, *irrigation* and *hydropower* are secondary purposes.

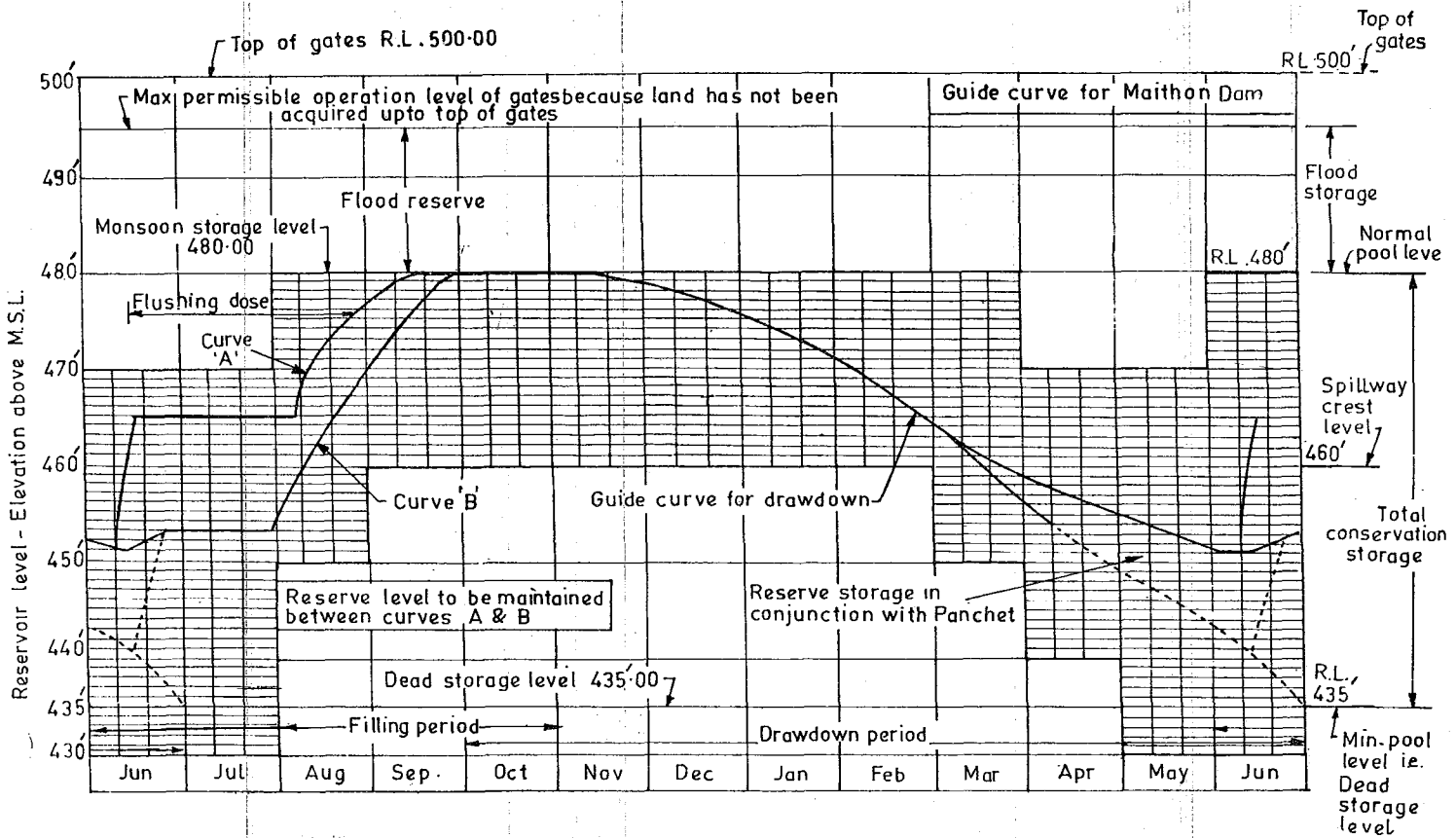


Fig. 18.23. A typical operating guide for Maithon Dam Reservoir.

18.13. Reservoir Sedimentation

Every river carries certain amount of sediment load. The sediment particles try to settle down to the river bottom due to the gravitational force, but may be kept in suspension due to the upward currents in the turbulent flow which may overcome the gravity force. Due to these reasons, the river carries fine sediment in suspension as suspended load, and larger solids along the river bed as bed load. When the silt laden water reaches a reservoir in the vicinity of a dam, the velocity and the turbulence are considerably reduced. The bigger suspended particles and most of the bed load, therefore, gets deposited in the head reaches of the reservoir. Fine particles may travel some more distance and may finally deposit farther down in the reservoir, as shown in Fig. 18.24. Some very fine particles may remain in suspension for much longer period, and may finally escape from the dam along with the water discharged through the sluiceways, turbines, spillway, etc.

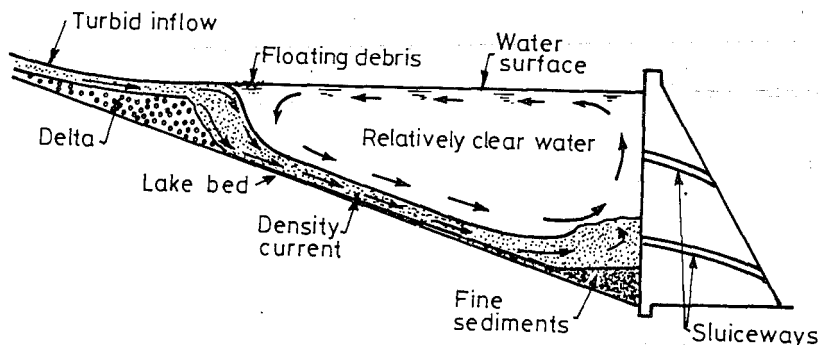


Fig. 18.24. Sediment accumulation in a typical reservoir.

The deposition of sediment in the reservoir is known as 'Reservoir Silting' or 'Reservoir Sedimentation'.

The deposition of the sediment will automatically reduce the water storing capacity of the reservoir, and if this process of deposition continues longer, a stage is likely to reach when the whole reservoir may get silted up and become useless.

Moreover, with the passage of time, the reservoir capacity will go on reducing. Thus, if today at the time of construction, a reservoir can store 10,000 cubic metres of water, tomorrow say after five years, it may be able to store only say 8,000 cubic metres of water. Therefore, in order to see that the capacity does not fall short of requirement ever during the design period, we must take this silting into account. The total volume of silt likely to be deposited during the designed life period of the dam is, therefore, estimated; and approximately that much of volume is left unused to allow for silting, and is known as **dead storage**. The remainder is known as the **effective storage** or the **live storage**. The dead storage generally varies between 15 to 25% of the total capacity. For example in Bhakra dam, the gross capacity of the dam is 9,344 million cubic metres and the dead storage provided is 2,054 million cubic metres. All the outlets fetching water from the reservoir are provided above the dead storage level.

The importance of this silting can be understood by considering the following example :

Let the total capacity of a reservoir be 30 million cubic metres and the provision of dead storage be 6 million cubic metres. Let the average volume of sediment deposition be 0.15 million cubic metres per year. Then it is evident, that the dead storage will be filled up in $\frac{6}{0.15} = 40$ years, and the total storage in about $\frac{30}{0.15} = 200$ years.

Hence, the usefulness of this reservoir would start reducing after 40 years, and after 200 years it would be nothing but a collection of sand and sediment with no water in it, provided the siltation rate remains constant at $0.15 \text{ M.m}^3/\text{yr}$.

18.13.1. Density Currents. In a reservoir, the coarser sediment settles down along the bottom of the reservoir, as the muddy flow approaches the reservoir; while the finer sediment usually remains in suspension, and moves in a separate layer than the clear reservoir water, as shown in Fig. 18.24. This layer of water, containing the fine sediment, moves below the upper clearer reservoir water, as a *density current*, since its density is slightly more than the density of the main body of the reservoir water. *Because of their density difference, the water of the density current does not mix easily with the reservoir water, and maintains its identity for a considerable time.* The density current can thus be removed through the dam sluiceways, if they are located properly and at the levels of the density current. A lot of sediment load can, thus, be passed out of the reservoir, if it is possible to locate the dam outlets and sluiceways in such a fashion, as to vent out the density currents. Trap efficiency of reservoirs may thus be decreased by about 2 to 10%, if it is possible to vent such density currents through the outlets and sluiceways of the dams.

18.13.2. Trap Efficiency. Now, we introduce another very important term called Trap efficiency. Trap efficiency is defined as the percentage of the sediment deposited in the reservoir even in spite of taking precautions and measures to control its deposition.

Therefore, Trap Efficiency (η)

$$= \frac{\text{Total sediment deposited in the reservoir}}{\text{Total sediment flowing in the river}} \quad \dots(18.13)$$

Most of the reservoirs trap 95 to 100% of the sediment load flowing into them. Even if various feasible silt control measures are adopted, it has not been possible to reduce this trap efficiency below 90% or so.

18.13.3. Capacity Inflow Ratio. The ratio of the reservoir capacity to the total inflow of water in it, is known as the capacity-inflow ratio. It is a very important factor, because the trap efficiency (η) has been found to be a function of capacity-inflow ratio i.e.

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right) \quad \dots(18.14)$$

The graph obtained for the existing reservoirs between trap efficiency and log of $\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$ has been found to be of the type shown in Fig. 18.25.

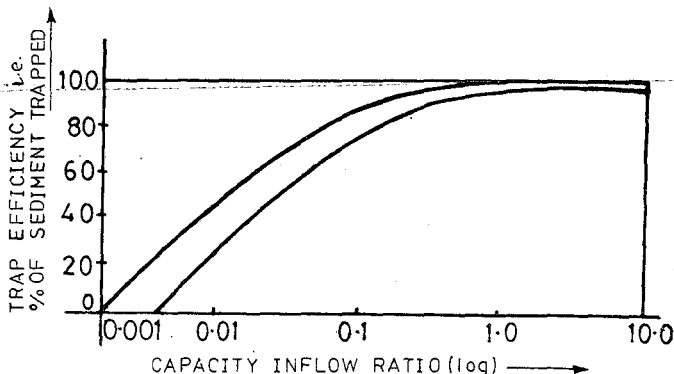


Fig. 18.25.

It is evident from the above curve, that if capacity reduces (with constant inflow), trap efficiency reduces, and hence, lesser sediment is trapped. Therefore, the silting rate in the reservoir shall be more in the beginning, and as its capacity reduces due to silting, the silting rate will reduce. Hence, the complete reservoir-silting may take longer period.

It can also be concluded that for small reservoirs (having small capacity) on large rivers (having large inflow rates), the trap efficiency is extremely low, because the capacity inflow ratio is very small. Such reservoirs silt very little and most of their sediment is passed downstream. On the other hand, large reservoirs on smaller rivers shall silt tremendously and almost complete deposition of sediment may take place.

18.13.4. Silting of Power Reservoirs. In case of reservoirs constructed solely for the purpose of power generation, the silting is comparatively less important. This is because of the fact, that for the proper and efficient functioning of a power reservoir, only a certain minimum head is necessary. This head remains available even after some silt gets deposited. So, only if sufficient water required for power generation remains available, the reservoir's efficiency remains unaffected by silting. But due to silted water, the abrasion of the blades of the turbines may occur very soon, and power production may be stopped over a considerable length of time.

18.13.5. Silting Control in Reservoirs. In order to increase the life of a reservoir, it is necessary to control the deposition of sediment. Various measures are undertaken in order to achieve this aim. The various methods which are adopted can be divided into two parts :

- (1) Pre-constructing measures ; and (2) Post-constructing measures.

These measures are discussed below :

(1) **Pre-constructing measure.** They are those measures which are adopted before and during the execution of the project. They are innumerable below :

(a) *Selection of Dam Site.* The silting depends upon the amount of erosion from the catchment. If the catchment is less erodible, the silting will be less. Hence, the silting can be reduced by choosing the reservoir site in such a way as to exclude the run off from the easily erodible catchment.

(b) *Construction of the Dam in Stages.* The design capacity plays an important role in the silting of a reservoir. When the storage capacity is much less than the average annual runoff entering the reservoir, a large amount of water will get out of the reservoir, thereby, reducing the silting rate compared to what it would have been if the entire water would have been stored. Therefore, the life of a reservoir can be prolonged by constructing the dam in stages. In other words, first of all, the dam should be built lower, and raised subsequently when some of its capacity gets silted up.

(c) *Construction of Check Dams.* The sediment inflow can be controlled by building check dams across the river streams contributing major sediment load. These are smaller dams and trap large amounts of coarser sediments. They are quite expensive.

(d) *Vegetation Screens.* This is based on the principle that vegetations trap large amounts of sediment. The vegetation growth is, therefore, promoted at the entrance of the reservoir as well as in the catchment. These vegetative covers, through which flood waters have to pass before entering the reservoirs, are known as vegetation screens, and provide a cheap and a good method of silt control.

(e) *Construction of Under-sluices in the Dam.* The dam is provided with openings in its base, so as to remove the more silted water on the downstream side.

The sediment concentration will be more at some levels than at others. Therefore, sluices are located at the levels of higher sediment concentration. The method in itself, is not sufficient because the water digs out a channel behind the sluice for movement and leaves most of the sediment undisturbed. Therefore, this is simultaneously supplemented with mechanical loosening and scouring of the neighbouring sediment in order

to increase its effectiveness. But to provide large sluices near the bottom of the dam, is again a structural problem. The use of this method is, therefore, limited.

(2) **Post-constructing Measures.** These measures are undertaken during the operation of the project. They are given below:

(1) **Removal of Post Flood Water.** The sediment content increases just after the floods ; therefore, attempts are generally made not to collect this water. Hence, the efforts should be made to remove the water entering the reservoir at this time.

(2) **Mechanical Stirring of the Sediment.** The deposited sediment is scoured and disturbed by mechanical means, so as to keep it in a moving state, and thus, help in pushing it towards the sluices.

(3) **Erosion Control and Soil Conservation.** This includes all those general methods which are adopted to reduce erosion of soil and to make it more and more stable. This method is the most effective method for controlling siltation, because when the soil erosion is reduced, the sedimentation problem is reduced automatically. But the methods of treating the catchment in order to minimise erosion are very costly. It has been estimated that the investment required for treating 16% of the Indian catchment area is Rs. 1,000 crores. In India, only 1.5% of the catchment area has been treated to minimise silting.

Example 18.16. The following information is available regarding the relationship between trap efficiency and capacity inflow ratio.

Capacity inflow ratio	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Trap efficiency percent	87	93	95	95.5	96	96.5	97	97	97	97.5

Find the probable life of the reservoir with an initial reservoir capacity of 30 million cubic metres, if the average annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 2,00,000 tonnes. Assume a specific weight of the sediment equal to 1.2 gm per c.c. The usual life of the reservoir will terminate when 80% of its initial capacity is filled with sediment.

Solution.

Average annual sediment inflow = 2,00,000 tonnes

$$= 2 \times 10^5 \text{ tonnes} = 2 \times 10^{11} \text{ gm}$$

Volume of average annual sediment inflow

$$= \frac{2 \times 10^{11}}{1.2} \text{ c.c.} = \frac{2 \times 10^{11}}{1.2 \times 10^6} \text{ m}^3$$

$$= \frac{0.2}{1.2} \times 10^6 \text{ cubic metres} = \frac{1}{6} \text{ million cubic metre} = \frac{1}{6} \text{ M.m}^3$$

Initial Reservoir Capacity = 30 M.m³

Annual flood inflow = 60 M.m³

Let us assume that 20% of the capacity, i.e. 6 M.m³ is filled up in the first interval.

$$\text{Capacity inflow ratio at the start of the interval} = \frac{30}{60} = 0.5$$

$$\text{Trap efficiency at the start of the interval} = 0.96.$$

$$\text{Capacity inflow ratio at the end of the interval} = \frac{24}{60} = 0.4$$

$$\text{Trap efficiency at the end of interval} = 0.955$$

$$\text{Average trap efficiency during the interval} = \frac{0.96 + 0.955}{2} = 0.9575.$$

Volume of sediment deposited annually till the 20% capacity is filled

$$= \frac{1}{6} \times 0.9575 \text{ M.m}^3$$

∴ No. of years during which 20% of the capacity, i.e. 6 M.m³ shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9575} \text{ years} = \frac{36}{0.9575} = 37.6 \text{ years}$$

Similarly, in the 2nd interval

$$\text{Capacity inflow ratio at the start} = \frac{24}{60} = 0.4$$

$$\text{Capacity inflow ratio at the end} = \frac{18}{60} = 0.3$$

$$\text{Trap efficiency at the start} = 0.955$$

$$\text{Trap efficiency at the end} = 0.95$$

$$\text{Average trap efficiency} = 0.9525.$$

∴ No. of years during which the next 20% of capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9525} = \frac{36}{0.9525} = 37.8 \text{ years.}$$

Similarly, in the 3rd interval

$$\text{Capacity inflow ratio at start} = \frac{18}{60} = 0.3$$

$$\text{Capacity inflow ratio at the end} = \frac{12}{60} = 0.2$$

$$\text{Trap efficiency at the start} = 0.95$$

$$\text{Trap efficiency at the end} = 0.93$$

$$\text{Average trap efficiency during the interval} = 0.94$$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.94} = \frac{36}{0.94} = 38.3 \text{ years.}$$

Similarly, in the 4th interval

$$\text{Capacity inflow ratio at the start} = 12/60 = 0.2$$

$$\text{Capacity inflow ratio at the end} = 6/60 = 0.1$$

$$\text{Trap efficiency at the start} = 0.93$$

$$\text{Trap efficiency at the end} = 0.87$$

$$\text{Average trap efficiency during the interval} = 0.90$$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9} = \frac{36}{0.9} = 40 \text{ years.}$$

Total probable life till 80% capacity gets filled up

$$= 37.6 + 37.8 + 38.3 + 40.0 = 153.7 \text{ years. Ans.}$$

The above calculations of dividing the entire capacity into intervals (20% each in the above case) can also be carried out in a tabular form, as shown below in Table 18.28.

Table 18.28

Capacity		Capacity/inflow Col. (2) $= \frac{60 \text{ Mcum}}{60 \text{ Mcum}}$	Trap efficiency η	Av. Trap eff. η_{av} during the interval	Sediment trapped per year; Col. (5) \times Av. annual sediment inflow = Col. 5 $\times \frac{1}{6}$ M.cum.	Years reqd. to fill up 20% capacity (6 Mcum.) $= \frac{6}{\text{Col. (6)}}$ in years
%	Capacity Vol. in M. cum.					
(1)	(2)	(3)	(4)	(5)	(6)	(7)
100	30	0.5	0.96	0.9575	0.1596	36.6
80	24	0.4	0.955	0.9525	0.1588	37.8
60	18	0.3	0.95	0.94	0.1566	38.3
40	12	0.2	0.93	0.90	0.15	40.0
20		0.1	0.8			
						$\Sigma = 153.7 \text{ yrs}$

Example 18.17. A proposed reservoir has a capacity of 500 ha-m. The catchment area is 125 km^2 , and the annual streamflow averages 12 cm of runoff. If the annual sediment production is 0.03 ha-m/km^2 , what is the probable life of the reservoir before its capacity is reduced by 10% of its initial capacity by sedimentation? The relationship between trap efficiency η (%) and capacity inflow ratio C/I , is as under :

C/I	0.01	0.02	0.04	0.06	0.08	0.1	0.2	0.3	0.5	0.7
$\eta\%$	43	60	74	80	84	87	93	95	96	97

(U.P.S.C., Civil Services, 1987)

Solution. Av. annual streamflow = 12 cm of runoff

$$\text{Area of catchment} = 125 \text{ km}^2 = 125 \times 10^6 \text{ m}^2$$

$$\therefore \text{Annual flood inflow} = (125 \times 10^6) \cdot \frac{12}{100} \text{ m}^3$$

$$= 15 \times 10^6 \text{ m}^3 = 15 \text{ M.m}^3 (\text{Mcum}).$$

$$\text{Annual sediment inflow} = 0.03 \text{ ha-m/km}^2 \text{ of the catchment}$$

$$= 0.03 \times 125 \text{ ha-m} = 0.03 \times 125 \times 10^4 \text{ m}^3 = 3.75 \times 10^4 \text{ m}^3$$

$$= \frac{3.75}{100} \times 10^6 \text{ m}^3 = 0.0375 \text{ M-m}^3 (\text{Mcum}).$$

It means that 0.0375 Mcum of sediment flows every year into the dam/reservoir site, but the quantum of this, which is trapped in the reservoir, depends upon the average trap efficiency (η) during that year, and this trap efficiency, in turn, depends upon the capacity/inflow ratio.

In the question, the total capacity to be filled up by sediment is 10% of the initial reservoir capacity,

$$\text{i.e. } 10\% \times 5 \text{ Mcum} = 0.5 \text{ Mcum}.$$

Now, we have to calculate the time during which this 0.5 Mcum of sediment will get deposited in the reservoir, as follows :

Capacity of the reservoir at the start = 5 Mcum

Capacity of the reservoir at the end

(i.e., when 0.5 Mcum of sediment is filled up) = 4.5 Mcum

$$\therefore \text{Capacity/inflow at the start} = \frac{5 \text{ Mcum}}{15 \text{ Mcum}} = 0.333$$

$$\eta \text{ at start} = 95\%. \quad \text{Capacity/inflow at the end} = \frac{4.5}{15} = 0.30$$

$$\eta \text{ at the end of the interval} = 95\%. \quad \text{Average } \eta = 95\%$$

\therefore Sediment load trapped/yr.

$$= 0.0375 \times 95\% = 0.035625$$

\therefore No. of years during which 0.5 Mcum of sediment will get trapped

$$= \frac{0.5}{0.035625} \text{ years} = 14.04 \text{ years ; say 14 years.}$$

Hence, after 14 years, 10% reservoir capacity will get filled up. Ans.

18.14. Estimating Sediment Load likely to Enter a Proposed Reservoir

The quantum of sediment flowing into a dam reservoir along with river runoff primarily depends upon the erosion characteristics of its catchment and the characteristics of the rainfalls that produce the run off entering the reservoir.

The sediment is basically produced by rains by the process of sheet erosion. The flowing water is the most active agent for erosion of soil from the land. Other agents like *wind*, *gravity*, *ice* and *human activities* do help in the erosion process. The rain drops in itself, loosen the soil particles and break the soil lumps. The action of flowing sheet of water on the land surface helps in eroding the top soil from the ground surface and transport it down to the channels. The erosion caused by rainfall and runoff, thus, constitutes of the following two parts :

(1) **Sheet erosion.** It includes the detachment of geological material from the land surface by the impact of raindrops, and its subsequent removal by overland flow; and

(2) **Channel erosion.** It includes river bank erosion and transportation of the materials by concentrated flow.

Human activities, like *overgrazing of grass land*, *cutting of forests*, *forest fires*, *ploughing of land*, and various *mining & other excavations* etc. have magnified the problem of water erosion in river channels.

18.14.1. Factors Affecting the Erodibility of a Soil. The factors which effect the erodibility of a soil are given below :

(i) **Particle size of soil.** Larger the size of soil particles, the lesser would be its chances for erosion.

(ii) **Land slope.** The greater is the land slope, the greater is the action of erosive agents, the optimum being at 40° slope.

(iii) **Vegetation.** The thicker is the vegetation cover over a soil, the lesser will be the scope of erosion of the soil from the area.

(iv) **Presence of Salt and Colloidal Matter in the Soil.** The binding materials like kaolinite, montmorillonite, biotite, etc., do help in increasing the force of cohesion between the soil particles, thereby reducing the erodibility of the soil.

(v) **Moisture Content of Soil.** The greater is the moisture content of the soil, the lesser is the scope of its erosion.

(vi) **Soil compaction.** The higher is the compaction of soil, the lesser is the chance of its erosion.

(vii) **Soil Properties.** The characteristics of the soil, such as soil texture, structure, stratification, permeability, composition etc. do affect the soil binding, which in turn will neutralise the force of weathering agents.

(viii) **Human Activities.** Human activities on the land like mining, agricultural operations, construction of projects, land use, etc. do increase the erosion from the given land.

(ix) *Rainfall characteristics.* The intensity, duration, quantity and distribution of rainfall over space and time are some of the important factors that affect sediment yield.

18.14.2. Estimation of Sheet Erosion. The sediment yield of a reservoir basically depends on sheet erosion which can be estimated by the following empirical equations :

(1) *Musgrave Equation.* Musgrave (1947) suggested the following equation to compute the annual gross sheet erosion from a catchment, on the basis of 19 widely scattered research stations in USA :

$$E = CR \cdot (S_0/10)^{1.35} (L/72)^{0.35} (P_{30}/1.25)^{1.75} \quad \dots(18.15)$$

where E = Erosion of soil lost from the catchment in inches/year

C = the *soil erosion rate*, which varies from 0.43 to 0.53 inch/year depending on the soil type (which depends on texture and permeability of soil)

R = the *cover factor*, which varies from 0.95 for poorly covered land to 0.10 for row crops

S_0 = Land slope in percentage, the default being 10%

L = Length of the land slope in feet

P_{30} = The max. rainfall in inches having 30 min. duration and of 2 year frequency.

It was also stipulated that the erosion value of E computed above, in inch/year, can be multiplied by 150 to obtain the erosion value in ton/year/acre.

(2) *Universal Equation.* Agriculture Research Service of U. S. Deptt. of Agriculture developed a universal equation (1961) to predict erosion from **small catchments**. This universal equation is given as :

$$E_a = R_f \cdot K \cdot (LS) C_m \cdot P \quad \dots(18.16)$$

where E_a = Average soil loss in ton/acre/year.

(i) R_f is **Rainfall-run off factor**. Its value takes into account the effect of rain drop impact as well as the resulting amount and rate of run off. It should also include the cumulative effects of many moderate sized storms as well as the effects of occasional severe storms.

The value of R_f is generally taken as equal to EI (**rainfall erosion index**), which is computed as :

$$E \cdot I = E \times I_{30} \quad \dots(18.17)$$

where E = storm energy in 100 ft-ton/acre/inch

I_{30} = Max. 30-minute intensity in inch/hr.

The value of E is further computed as :

$$E = \frac{1}{100} \left[916 + 331 \log I \right] \quad \dots(18.18)$$

where E = Storm energy in 100 ft-ton per acre per inch.

I = Storm intensity of the given storm with a limit of 3 inch/hr, since median drop size does not continue to grow beyond this limit.

The EI value can, thus, be computed by eqn (18.17) for a given storm. For a specified period, the individual storm EI values can be summed up, which provide a numerical measure of the erosive potential of the rainfall within that period (one year). *In this manner, the average annual total of the storm EI values in a particular area is obtained, which is called the rainfall-erosion index for that area, which equals R_f .*

- (ii) K is **Soil erodibility factor**. Its value is determined experimentally for the given soil type. Representative values for different types of soils have been worked out and listed for different regions of USA by Soil Conservation Service of USA, as shown in Table 18.29.

Table 18.29. Computed K Values for Soils on Erosion Research Stations (After Wischmeier and Smith, 1978).

Soil	Source of Data	Computed K
Dunkirm silt loam	Geneva, NY	0.69 ^a
Keene silt loam	Zanesville, OH	0.48
Shelby loam	Bethany, MO	0.41
Lodi loam	Blacksburg, VA	0.39
Fayette silt loam	LaCrosse, WI	0.38 ^a
Cecil sandy clay loam	Watkinsville, GA	0.36
Marshall silt loam	Clarinda, IA	0.33
Ida silt loam	Castana, IA	0.33
Mansic clay loam	Hays, KS	0.32
Hagerstown silty clay loam	State College, PA	0.31 ^a
Austin clay	Temple, TX	0.29
Mexico silt loam	McCredie, MO	0.28
Honeoye silt loam	Marcellus, NY	0.28 ^a
Cecil sandy loam	Clemson, SC	0.28 ^a
Ontario loam	Geneva, NY	0.27 ^a
Cecil clay loam	Watkinsville, GA	0.26
Boswell fine sandy loam	Tyler, TX	0.25
Cecil sandy loam	Watkinsville, GA	0.23
Zaneis fine sandy loam	Guthrie, OK	0.22
Tifton loamy sand	Tifton, GA	0.10
Freehold loamy sand	Marlboro, NJ	0.08
Blath flaggy silt loam with surface stones > 2 inches removed	Arnot, NY	0.05 ^a
Albia gravelly loam	Beemerville, NJ	0.03

^a Evaluated from continuous fallow. All others were computed from rowcrop data.

For soils containing less than 70% of silt and very fine sand, K can be computed as :

$$K = \frac{1}{100} \left[2.1 M^{1.14} (10^{-4}) (12 - a) + 3.25 (b - 2) + 2.5 (c - 3) \right] \quad \dots (18.19)$$

where M = particle size parameter, defined as percent silt & very fine sand (size 0.1 mm-0.002 mm) times the quantity (100-percent clay)

a = percent organic matter

b = soil texture code used in USDA soil classification

c = profile permeability class

[Note : When the silt fraction does not exceed 70%, erodibility varies approximately as the 1.14 power of M , but addition of organic matter content, soil structure, and profile-permeability class as done in Eqn. (18.19) improves the prediction accuracy.]

(iii) **(LS) = Topographic Factor.** It is the ratio of soil loss per unit area from a field slope to that from a 72.6 ft length of uniform 9% slope under otherwise identical conditions. For a specified slope and its length, *LS* can be computed as :

$$(LS) = \left(\frac{\lambda}{72.6} \right)^m (65.41 \sin^2 \theta + 4.56 \sin \theta + 0.065) \quad \dots(18.20)$$

where λ = slope length in ft

θ = angle of slope

$M = 0.5$ if the per cent slope is 5 or more

$m = 0.4$ on slopes of 3.5 to 4.5%

$m = 0.3$ for slopes of 1 to 3%

$m = 0.2$ for uniform slopes of less than 1%.

(iv) **C_m = Soil Cover and Management Factor.** It measures the combined effect of all inter-related cover and management variables, including the type of vegetation, plant spacing, the stand, the quality of growth, crop sequence, tillage practices, crop residues, incorporated residues, land use residues, fertility treatment, etc.

Values of C_m for pasture, range, idle land, and woodland for a combination of cover conditions are given in Table 18.30.

Table 18.30. Factor C_m for permanent Pasture, Range, and Idle Land
(After Wischmeier and Smith 1978).

Vegetative canopy		Cover that contracts the soil surface						
Type and height ^b	Percent cover ^c	Type ^d	Percent ground cover					
			0	20	40	50	80	95+
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.091	0.043	0.011
Tall weeds or short brush with the average drop fall height of 20 in.	25	G	0.36	0.17	0.09	0.038	0.013	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011
	50	G	0.26	0.13	0.07	0.035	0.120	0.003
		W	0.26	0.16	0.11	0.076	0.039	0.001
	75	G	0.17	0.10	0.06	0.032	0.011	0.003
		W	0.17	0.12	0.09	0.068	0.038	0.011
Appreciable brush or bushes, with the average drop fall height of 6 ½ ft	25	G	0.40	0.18	0.09	0.040	0.013	0.003
		W	0.40	0.22	0.14	0.087	0.042	0.011
	50	G	0.34	0.16	0.08	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.082	0.042	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.078	0.040	0.011
Trees, but no appreciable low brush Average drop fall height of 13 ft	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.089	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.087	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.084	0.041	0.011

^a The listed *C* values assume that the vegetation and mulch are randomly distributed over the entire area.

^b Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to the drop fall height and is negligible if fall height exceeds 33 ft.

^c Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

^d G: cover at the surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 in. deep.

W: cover at the surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

- (v) P = **Support Practice Factor**. Its value depends upon crop land practice such as contour tillage, *strip cropping on the contour*, and *terrace systems*. Values of P have been given for each of these practices by Wischmeier and Smith (1978). In general, tillage and planting on the contours reduce erosion. Table 18.31 gives P values for contouring.

Table 18.31. P values (Eq. 18.16) and Slope-Length Limits for contouring (Wischmeier & Smith)

Land Slope %	P Value	Maximum Length* (feet)
1 - 2	0.60	400
3 - 5	0.50	300
6 - 8	0.50	200
9 - 12	0.60	120
13 - 16	0.70	80
17 - 20	0.80	60
21 - 25	0.90	50

* Limit may be increased by 25% for residue cover after crop seedlings will regularly exceed 50%.

Since it is difficult to get proper data for the use of Eq. (18.16) for a developing country like India, this equation is not popular in developing countries.

18.14.3. Sediment Measurement by Sample Recorder. The sediment produced by sheet erosion from a catchment may not always reach at the point of measurement; i.e. the site of dam reservoir. Some part of sediment may be deposited en-route. *The ratio between the yield of sediment at the measuring site and the gross erosion in the catchment is called the sediment delivery ratio.*

Thus, the **sediment yield** is the gross sediment yield minus the quantity of sediment deposited en route. The sediment yield, in fact, is important, since it is this sediment which will get deposited in the reservoir, affecting its useful life.

The *sediment yield of a reservoir* can be calculated either by using an appropriate empirical equation, out of the ones developed by various investigators; or by developing a rational appropriate relation between inflow and sediment, on the basis of actual measurements of sediment load at the site of the reservoir. The continuous measurements of suspended load and bed load at the reservoir site for a number of years will help in developing a rational relation (either a mathematical equation or a graphical curve) between sediment and the inflow into the reservoir. We will first of all discuss the method of the actual measurement of sediment and developing an appropriate relation between sediment yield (q_s) and the river discharge (Q) at the given site.

18.14.3.1. Measuring suspended sediment load. Sediment samplers are used to measure suspended sediment at a given site on the river to obtain the most reliable results of sediment yield. The bed load should also be calculated either by using the available empirical equations*, or on an adhoc basis of 2.5-25% of the suspended load, as to calculate the total sediment load (sum of suspended load and bed load).

A typical sediment sampler used in India is shown in Fig. 18.26. A depth integrating hand sampler used for small streams is also shown in Fig. 18.27.

* Please see article 4.9.

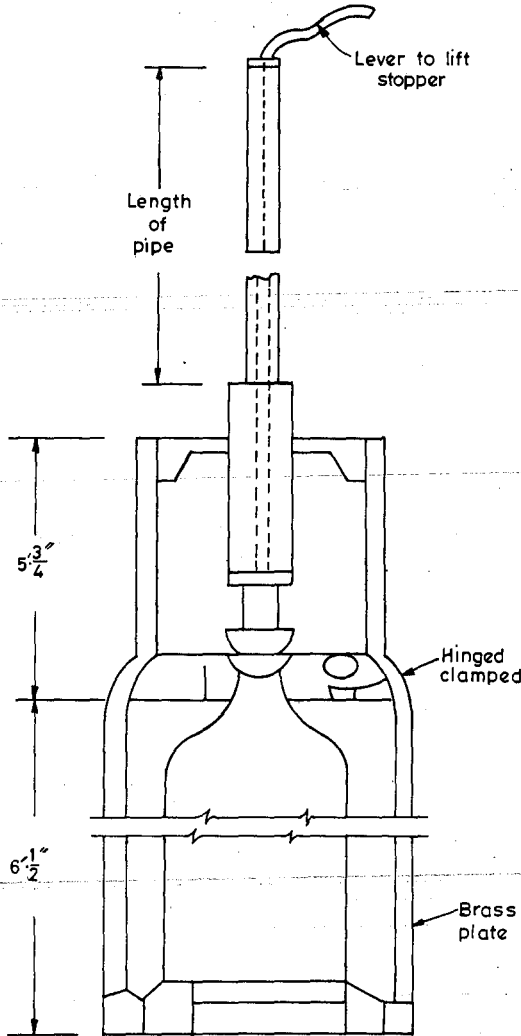


Fig. 18.26. Line diagram of a typical sediment sampler (Punjab bottle sampler).

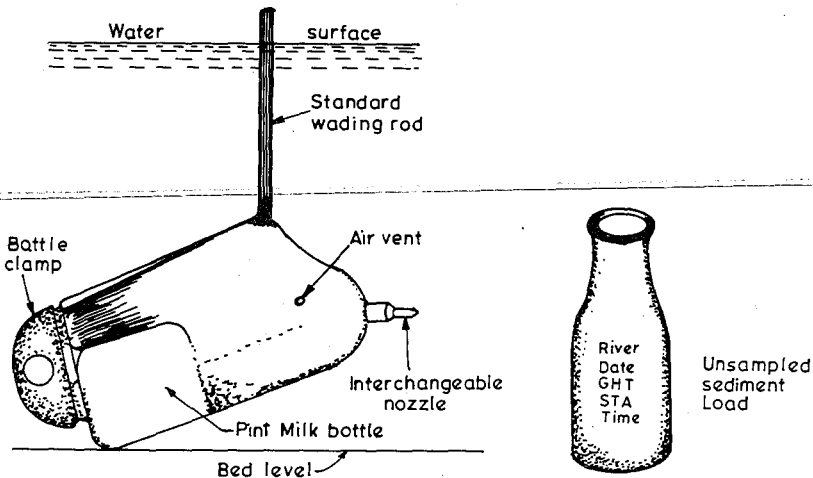


Fig. 18.27. AUS.DH-48 depth-integrating hand sampler for small streams.

The sampler is taken into the stream to a depth of $0.6y$ below the water surface, or at two depths at $0.2y$ and $0.8y$ to collect samples of sedimented water in the sampler bottle. The collected sample of sedimented water is then analysed in the laboratory either by a *gravimetric method*, or by *hydrometric method* to determine the quantum of coarse sediment (particle size higher than 0.2 mm), *medium size sediment* (particle size between 0.075 mm to 0.2 mm) and of *fine sediment* (particle size less than 0.075 mm), separately*. Their sum will indicate the total sediment load present in the given volume of water sample. Sediment load present in the water sample is then expressed in ppm (parts for million) as :

$$\text{Sediment load in ppm} = \frac{\text{Dry mass of sediment}}{\text{Total mass of original sample including the mass of sediment \& of water}} \times 10^6 \quad \dots(18.21)$$

This value can be converted into t/day by multiplying the average unit wt. of sediment (say 1.2 t/m^3) with the total volume of daily inflow in m^3 .

When a large number of such sample records become available for the given site, then a curve can be plotted between the sediment load (q_s) in t/day on x-axis and daily discharge in m^3/s on y-axis, as to obtain a curve known as *sediment rating curve*, as shown in Fig 18.28. Such a plotting is usually done on a log-log paper.

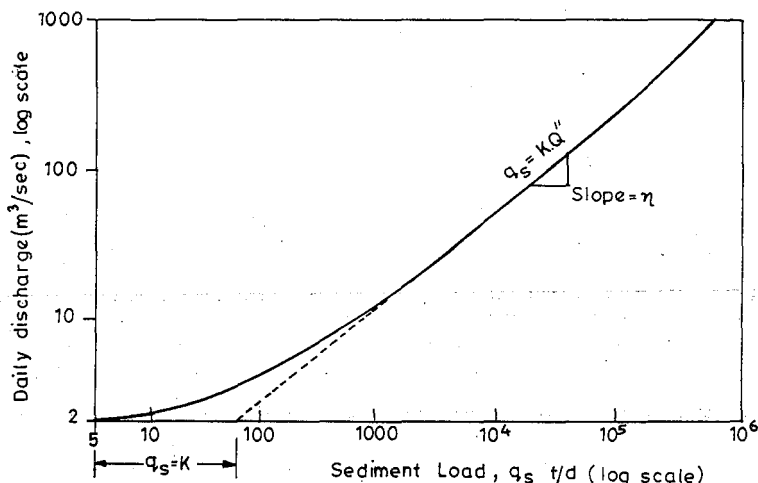


Fig. 18.28. A typical sediment rating curve.

When a standard sediment rating curve is established for a given site, the sediment yield (q_s) can be read out by simply knowing the discharge rate only. However, care must be taken to see that for different seasons of the year, different curves will have to be developed, since the sediment yield of a basin may vary with the season. The curve, for say March (non-monsoon period) may be entirely different from the curve for the month of say August (monsoon season).

* The sample collected by the sampler is first passed through a BSS-100 sieve and the coarse particles retained are taken out and oven dried. Thus, the quantity of coarse sediment (higher than 0.2 mm size) is obtained. Sedimented water passing 200μ sieve is allowed to stand for 20 minutes, so that the finer particles settle down. The settled mass is removed by the process of decantation (pouring out water from the settled tank). The settled residue is dried and weighed, as to get the mass of fine sediment (0.075 mm – 0.2 mm). Poured water contains sediment of still finer particles. To isolate this, the sample is filtered through a filter paper, and the quantity retained therein is dried and weighed. This gives the mass of fine sediment (particle size < 0.075 mm). The summation of all the three masses will give the total mass of sediment.

The sediment rating curves can be used to compute daily sediment load for the given daily discharge values, and their summation can give us the monthly or the annual sediment load.

18.14.3.2. Mathematical equation for a sediment rating curve. The sediment rating curve (straight line portion) giving sediment load in tonnes/day (q_s) w.r. to daily discharge (Q) in m^3/sec can be expressed by a mathematical equation of the form :

$$q_s = K \cdot Q^n \quad \dots(18.22)$$

Taking log on both sides,

$$\log q_s = \log K + n \cdot \log Q$$

This equation is similar to the form

$$y = a + bx,$$

where $\log q_s$ is plotted on y-axis and $\log Q$ on x-axis.

$$n = b \quad \dots(18.23)$$

$$\log K = a \quad \dots(18.24)$$

A mathematical solution for such an equation can be obtained by the statistical **method of least squares**, where in the various known values of x (i.e. $\log Q$) and y (i.e. $\log q_s$) are analysed to estimate a and b values as :

$$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N \Sigma x^2 - (\Sigma x)^2} \quad \dots(18.25)$$

$$b = \frac{N \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2} \quad \dots(18.26)$$

By computing values of a and b , values of n and K become known to finally compute the relation between q_s and Q by Eq. (18.22).

The use of this method will become clear when we solve example 18.18.

Example 18.18. A reservoir has the following sediment and discharge data :

Year	81	82	83	84	85	86	87	88
Discharge ($M m^3$)	1430	3850	2050	6510	2880	1120	6050	2220
Sediment load ($M.t$) as measured by silt sampler	2.65	5.82	3.60	7.15	5.22	1.95	6.88	3.94

Calculate the average total sediment load/year/100 sq. km of the catchment at the site. Develop a regression relation and predict the total and observed sediment yield for the inflow of $3450 M.m^3$ for the year 1978. Take the catchment area at the site as 3050 sq. km. What is the total sediment yield for 100 years ? Assume bed load as 10% of suspended load.

Solution. Average sediment load measured by silt sampler

$$= \frac{2.65 + 5.82 + 3.6 + 7.15 + 5.22 + 1.95 + 6.88 + 3.94}{8}$$

$$= \frac{37.21}{8} \text{ Mt/year} = 4.651 \text{ Mt/yr.}$$

Since sediment load is measured by a silt sampler, the given values of sediment are of suspended load.

Now, assume Bed load = 10% of suspended load

∴ Total load = Suspended load + Bed load
∴ Total sediment load = 1.1 × 4.651 = 5.116

or $(q_s) = 5.116 \text{ Mt/yr.}$

Sediment deposited in a reservoir consolidates gradually due to the increasing silt load on it every year and the wt. of water above it. Assuming the average sediment unit weight (sediment in a reservoir consists of sand, silt and clay in water) as 1.2 t/m^3 , we have

∴ Total silt load/year = 5.116 Mt

$$q_s = \frac{5.116}{1.2} \text{ M} \cdot \text{m}^3 = 4.263 \text{ M m}^3$$

Catchment area = 3050 sq. km

∴ Sediment load/100 sq. km/year = $\frac{4.263 \text{ M m}^3}{3050} \times 100$
 $= 0.1397 \text{ M m}^3/100 \text{ sq. km/year Ans.}$

Total sediment yield in 100 years = $4.263 \text{ M m}^3/\text{yr.} \times 100 \text{ yr.} = 426.3 \text{ M m}^3 \text{ Ans.}$

In order to develop a regression type non-linear equation between total sediment load (q_s) and discharge (Q), represented as $q_s = K \cdot Q^n$, we carry out the required calculations in table 18.32 to evaluate the values of a and b by Eqn. (18.25) and (18.26), as :

Table 18.32. Computations to develop Equation $y = a + bx$ by the method of least squares.

year	Discharge $Q (\text{M} \cdot \text{m}^3)$	Suspended Sediment measured in Mt	Total sediment i.e. q_s in M m^3 $\text{col (3)} \times \frac{1.1^*}{1.2}$	$\log Q = x$ $\log \text{col (2)}$	x^2 $(\text{col 5})^2$	$\log q_s = y$ $\log \text{col (4)}$	$x \cdot y$ $\frac{\text{col (5)} \times \text{col (7)}}{\text{col (7)}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
81	1436	2.65	2.429	3.1553	9.9561	0.3855	1.2164
82	3850	5.82	5.335	3.5855	12.8555	0.7271	2.6070
83	2050	3.60	3.300	3.3118	10.9677	0.5185	1.7172
84	6510	7.15	6.554	3.8136	14.5433	0.8165	3.1138
85	2880	5.22	4.785	3.4594	11.9674	0.6799	2.3520
86	1120	1.95	1.788	3.0492	9.2977	0.2522	0.7690
87	6050	6.88	6.307	3.7818	14.3017	0.7998	3.0247
88	2220	3.94	3.612	3.3464	11.1981	0.5577	1.8663
			Σ	27.503	95.0875	4.7372	16.6664

* q_s (Total sediment in Mm^3) = $\frac{q_s \text{ in } \text{M} \cdot \text{t}}{1.2 \text{ t/m}^3} \times (1.1)^{**}$ ** Total sediment = 1.1 (Observed sediment by soil sampler)
 $= \left(\text{col (3)} \times \frac{1.1}{1.2} \right)$

Using
$$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N \Sigma x^2 - (\Sigma x)^2} \quad \dots(i.e. 18.25)$$

we get
$$a = \frac{4.7372 \times 95.0875 - 27.503 \times 16.6664}{8 \times 95.0875 - (27.503)^2}$$

or
$$a = \frac{-7.9275}{4.2850} = -1.8500$$

But $\log K = a = -1.8500 \quad \therefore K = 0.014$

Also
$$b = \frac{N \cdot \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2} \quad \dots(i.e. 18.26)$$

$$= \frac{8 \times 16.6664 - 27.503 \times 4.7372}{8 \times 95.0875 - (27.503)^2} = \frac{3.0440}{4.2850} = 0.7104$$

But $b = n = 0.7104$

Regression equation is hence given as :

$$q_s = K \cdot Q^n = 0.014 (Q)^{0.7104} \quad \text{Ans.}$$

Total sediment yield for the year 1978 having inflow of 3450 M m^3

$$= 0.014 (3450)^{0.7104} = 4.565 \text{ M m}^3$$

$$= 4.565 \text{ M m}^3 \times 1.2 \text{ t / m}^3 = 5.48 \text{ Mt} \quad \text{Ans.}$$

Observed or suspended sediment load for the year 1978

$$= \frac{5.48}{1.1} \text{ Mt} = 4.98 \text{ Mt} \quad \text{Ans.}$$

18.14.3.3. Empirical equations for total sediment yield. The following empirical equations have been developed by several investigators, for estimating the annual sediment yield of a reservoir.

(1) *Swami's Regression Equation.* Swamy and Garde (1977) have proposed a relation correlating the cumulative volume of sediment deposited in a reservoir with the cumulative volume of water inflow, and initial bed slope of the river, as

$$V_s = C \cdot B \cdot (V_{ci})^{0.94} (S_o)^{0.84} \quad \dots(18.27)$$

where V_s = cumulative vol. of sediment deposited in the reservoir in $\text{M} \cdot \text{m}^3$

C = Regression constant with safe value of the order of 1.16. However a value less than 1.16 may be adopted depending on the reservoir

B = Width of the reservoir at full reservoir level in m

V_{ci} = Cumulative vol. of inflow per unit width B of the reservoir

S_o = Bed slope of the river.

(2) *Jogelkar's Equation.* An equation proposed by Jogelkar (1960) is given as :

$$Q_s = 0.59 (A)^{-0.24} \quad \dots(18.28)$$

where Q_s = Annual silting rate in $\text{M} \cdot \text{m}^3$ per 100 sq. km of catchment area

A = Catchment area in sq km

(3) *Khosla's Equation.* Khosla (1953) has proposed the following empirical equation.

$$Q_s = 0.323 (A)^{-0.28} \quad \dots(18.29)$$

where Q_s = Annual siltation rate in $M \cdot m^3/100 \text{ sq km/year}$

A = Catchment area in sq. km.

This equation always gives lower estimate of sediment yield at a site.

(4) *Varshneys Equations.* Varshney and Raichur have proposed the following equations for calculating sediment yield for an ungauged basin.

(i) *Up to 130 sq. km catchment for mountainous rivers*

$$Q_s = 0.395 (A)^{-0.311} \quad \dots(18.30)$$

where Q_s = Annual sediment yield rate in $M \cdot m^3$ per 100 sq. km of catchment

A = catchment area in km^2 .

(ii) *Rivers draining plain area up to 130 sq. km*

$$Q_s = 0.392 (A)^{-0.302} \quad \dots(18.31)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(iii) *For area greater than 130 sq. km for North Indian catchment*

$$Q_s = 1.534 (A)^{-0.264} \quad \dots(18.32)$$

Q_s & A have the same meaning as in Eq. (18.30).

(iv) *For South Indian Rivers up to 130 sq. km*

$$Q_s = 0.46 (A)^{-0.468} \quad \dots(18.33)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(v) *For areas greater than 130 sq. km for South Indian catchments*

$$Q_s = 0.277 (A)^{-0.194} \quad \dots(18.34)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(5) *Using Known Data of Similar Catchment.* Sediment yield of an unmeasured watershed Q_{s_2} can be computed from sediment yield of measured catchment Q_{s_1} of similar topography, land cover and land use, on area proportion basis, as

$$Q_{s_2} = Q_{s_1} \left(\frac{A_2}{A_1} \right) \quad \dots(18.35)$$

Example 18.19. Estimate the sediment load in tonne at the proposed dam site in North India with the following data using various empirical equations :

Catchment area = 1830 sq. km

Width of reservoir at FRL = 560.0 m

River slope at the dam site = 0.006

Average inflows at the site are as follows :

Year	1982	1983	1984	1985	1986	1987	1988	1989	1990
Inflow $M \cdot m^3$	2210	1290	1640	1780	2150	1980	2540	1285	1620

Assume annual siltation rate per 100 sq. km from a similar catchment of 3050 sq. km to be $10.35 \text{ M} \cdot \text{m}^3/100 \text{ sq. km}$.

Solution. The sedimentation rate is worked out by using various equations as described in the previous article.

(1) *By Swamy's Regression Method* : The computation is carried out in Table 18.28 by using Eqn. (18.27) as :

$$V_s = C \cdot B (V_{ci})^{0.94} (S_0)^{0.84}$$

$$\text{where } C = 1.16$$

$$B = 560 \text{ m}$$

$$S_0 = 0.006$$

$$\therefore V_s = \text{cumulative vol. of sediment deposited in } \text{M} \cdot \text{m}^3$$

$$= 1.16 \times 560 (V_{ci})^{0.94} \times (0.006)^{0.84}$$

$$= 8.837 (V_{ci})^{0.94}$$

where V_{ci} is the cumulative vol. of annual inflows per unit width B of reservoir, over the given years.

The computations are carried out in Table 18.33, which is self explanatory.

Table 18.33. Computation of sediment Load by Swamy's Regression Method

S. No.	Year	Inflow in $\text{M} \cdot \text{m}^3$	Cumulative inflow in $\text{M} \cdot \text{m}^3$	$V_{ci} = \text{cumulative inflow per unit width } B \text{ of reservoir} = \frac{\text{col (4)}}{560}$	$V_s = 8.837 (V_{ci})^{0.94} = 8.837 (\text{col 5})^{0.94} \text{ M} \cdot \text{m}^3$
(1)	(2)	(3)	(4)	(5)	(6)
1	1982	2210	2210	3.946	32.11
2	1983	1290	3500	6.250	49.48
3	1984	1640	5140	9.179	71.01
4	1985	1780	6920	12.357	93.91
5	1986	2150	9070	16.196	121.10
6	1987	1980	11050	19.732	145.80
7	1988	2540	13590	24.268	177.11
8	1989	1285	14875	26.563	192.81
9	1990	1620	16495	29.455	212.48

$$\begin{aligned}
 Q_s &= \text{sedimentation rate per 100 sq. km} \\
 &= \frac{V_s \text{ (i.e. cumulative vol. of silt)}}{\text{No. of years (i.e. 9)}} \times \frac{100 \text{ sq. km}}{C \cdot A \text{ of } 1830 \text{ km}^2} \\
 &= \frac{212.48}{9} \times \frac{100}{1830}
 \end{aligned}$$

or $Q_s = 1.29 \text{ M m}^3/100 \text{ sq. km}$ Ans.

(2) By Jogelkar's Equation

$$\begin{aligned}
 Q_s &= 0.59 (A)^{-0.24} \\
 &= 0.59 (1830)^{-0.24} \\
 &= 0.10 \text{ M m}^3/100 \text{ sq. km} \quad \text{Ans.}
 \end{aligned}$$

(3) By Khosla's Equation

$$\begin{aligned}
 Q_s &= 0.323 (A)^{-0.28} \\
 &= 0.323 (1830)^{-0.28} \\
 &= 0.04 \text{ M m}^3/100 \text{ sq. km} \quad \text{Ans.}
 \end{aligned}$$

(4) By Varshneys Equation for Norths Indian catchment exceeding 130 sq. km.

$$\begin{aligned}
 Q_s &= 1.534 (A)^{-0.264} \\
 &= 1.534 (1830)^{-0.264} \\
 &= 0.211 \text{ M m}^3/100 \text{ sq. km} \quad \text{Ans.}
 \end{aligned}$$

(5) From similar catchment.

$$\begin{aligned}
 Q_{s_1} &= 4.56 \text{ M m}^3/100 \text{ sq. km} \\
 A_1 &= 3050 \text{ sq. km} \\
 A_2 &= 1830 \text{ sq. km} \\
 Q_{s_2} &= Q_{s_1} \times \left(\frac{A_2}{A_1} \right) \\
 &= 0.35 \times \left(\frac{1830}{3050} \right) \text{ M m}^3/100 \text{ sq. km} \\
 &= 0.21 \text{ M m}^3/100 \text{ sq. km} \quad \text{Ans.}
 \end{aligned}$$

18.15. Reservoir Sedimentation Studies on Existing Reservoirs

Sedimentation of storage reservoirs is a natural process, since large part of the silt eroded from the catchment and transported by the river, gets deposited on the bed of the reservoir. This causes reduction in the live as well as dead storage capacities of the reservoir. Progressive loss of capacity due to sediment accumulation results in reduced benefits and may even cause operational problems. *It, therefore, becomes necessary to monitor the sedimentation rates in the existing reservoirs at regular intervals, to help in planning and executing suitable remedial measures for controlling sedimentation in order to prolong the life of the reservoir and its benefits.*

Regular monitoring and updating of elevation-capacity curve of the reservoir immensely helps in better water management. With this aim, *conventional hydrographic surveys* are conducted at regular intervals at the existing reservoirs to determine the available capacities at different elevations, and to help compute the sedimentation volume at such regular intervals. Such conventional surveys will require computation

of water spread area at different water levels, and is quite a tedious and a costly process. **Remote sensing techniques** do offer a modern answer to the costly conventional surveys, as it offers a great potential for application in capacity evaluation of medium to large reservoirs. From the data provided by the remote sensing satellites, it has now been possible to compute loss of reservoir capacity due to sedimentation, and its distribution. The results obtained from this technique have been found to be quite comparable with those obtained from the costly and cumbersome conventional methods. One of the greatest advantage of this technique is that the capacity evaluation could be easily computed on yearly basis.

The methodology involved in this technique requires the use and analysis of the *satellite imageries* provided by the Remote Sensing satellites, which collect the data of the Earth surface features in different bands at regular intervals. In the case of Indian Remote Sensing Satellites IRS-1A and IRS-1B* (both identical satellites), this periodicity is 22 days for Indian sub-continent. These two satellites together are thus capable to provide us data of all our reservoirs at 11 days interval. The third satellite of this series IRS-1C has also been launched recently on 28.01.1996 and helps in providing better pictures even of cloud bound areas. IRS-1D has further been launched on 29.09.1997, through our first Indian rocket launcher.

Due to the water withdrawals from an existing reservoir, its water spread area goes on changing throughout the year. The reservoirs are generally full just after the monsoon period in October, and get depleted to almost dead storage/minimum drawdown level just before start of monsoon season in May or June, every year. The satellite data of various dates during the period from October to May, provide us an array of water spread areas between maximum water level (i.e. around the FRL) and the minimum reservoir level (i.e. around the dead storage level/minimum drawdown level). From the whole set of the satellite data, a few of them which are cloud free and of good quality and representative of the whole range of reservoir levels at close intervals, are selected for analysis.

The method of analysis depends upon the data products. The selected CCT's/FCC's of various dates are analysed for determining the water spread areas. The corresponding water levels are obtained from the daily gauge record of the reservoir. From these, the water volumes between two consecutive water levels are computed using Prismoidal or any appropriate formula. Volume of water below the minimum water level (as recorded by the satellite) and the "new zero" elevation, are estimated based upon the previous hydrographic surveys. In case, these informations are not available in the hydrographic survey data, then the elevation-area relationship obtained from the hydrographic surveys as well as the one obtained from the satellite data interpretation, should be extended to get the new zero elevation, and then the volume between the minimum mapped water level and this new zero level is estimated. After this, the cumulative water volume at each reservoir level is computed, and then the revised elevation-area-capacity curve is drawn. By comparing the original area-capacity curve or any other such curve, the total sediment volume and its distribution can be computed.

18.16. Observed Sedimentation Rates for Various Important Indian Reservoirs

The observed sedimentation rates for various important dam reservoirs in India are indicated in col. (6) & (7) of Table 18.34.

* IRS-1A was launched on 19.03.1988, and IRS-1B was launched on 23.08.1991.

** The minimum reservoir bed level is raised due to sedimentation, which is termed as the new zero.

Table 18.34. Annual Sedimentation Rates of Various Indian Reservoirs

S. No.	Name and Location of reservoir	Catchment area in 100 sq. km	Capacity of reservoir at FRL in $M \cdot m^3$	Surface area at MRL $M \cdot m^2$	Annual sediment rate in $ham/100$ sq. m	Annual volume of sediment deposit $M \cdot m^3$ $\frac{\text{col (6)}}{100} \times \text{col (3)}$	Dead storage capacity provided in $M \cdot m^3$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	Bhakra i.e. Govind Sagar (HP)	568	9351	169	6.00	34.08	95
2	Gandhi Sagar (MP)	226	7413	660	10.08	22.71	586
3	Hirakund (Orissa)	834	8146	725	3.89	32.44	2318
4	Lower Bhawani (TN)	61.5	929	—	4.10	2.52	0
5	Maithan (DVC)	63	1275	—	13.02	8.20	165
6	Matatila (UP)	207.5	883	—	3.50	7.26	50
7	Mayurkshi (WB)	18.6	617	—	20.09	3.74	68
8	Nizam Sagar (AP)	216.94	715	130	6.57	14.25	—
8	Panchat (DVC)	111	1475	153	9.92	11.01	170
10	Ramganga (UP)	30.76	2448	—	17.30	5.32	395
11	Sriram Sagar or Shivaji Sagar (AP)	8.19	3454	—	15.20	1.24	849
12	Tawa (MP)	—	2312	—	8.10	—	263

18.17. Reservoir Losses

Huge quantity of water is generally lost from an impounding reservoir due to *evaporation*, *absorption*, and *percolation*. Depending upon which, the following losses may occur from such a reservoir :

1. *Evaporation losses* ;
2. *Absorption losses* ; and
3. *Percolation losses* or *Reservoir leakage*.

These losses are discussed below.

18.17.1. Evaporation Losses. The evaporation losses from a reservoir depend upon several factors, such as : *water surface area*, *water depth*, *humidity*, *wind velocity*, *temperature*, *atmospheric pressure* and *quality of water* as discussed in article 7.34.2.3. The evaporation loss from a reservoir under the given atmospheric conditions can be easily estimated by measuring the standard pan evaporation and multiplying the same by the pan coefficient. The pan coefficients and various types of pans in use are given in article 7.34.3.4.

The evaporation losses become very significant in a hot and humid country like India ; and realistic estimation of these losses is quite important. These losses in fact vary from place to place and from season to season, and hence monthly values of these losses are usually determined. Typical average values of these losses for North and South India are given in table 18.35.

Table 18.35. Monthly Reservoir Evaporation Losses

Month	Losses in cm	
	North India	South and Central India
January	7	10
February	9	10
March	13	18
April	16	23
May	27	25
June	24	18
July	18	15
August	14	15
September	14	15
October	13	13
November	9	10
December	8	10
Total	172	182

On the basis of a review conducted on 130 sample reservoirs, the Central Water Commission, in 1990, has, however, estimated the *average annual evaporation loss to be 225 cm*; and the total water lost from all the existing reservoirs to be 27000 Mm³ per annum. What a tremendous waste of precious water!

In order to control such large scale wastage of water, several methods have been devised by engineers and scientists. All these methods are based upon the efforts made to reduce the evaporation rate from the surface of the water bodies by physical or chemical means, since the basic meteorological factors affecting evaporation cannot be controlled under normal conditions. *The following methods are generally used for evaporation control :*

1. Wind breakers
2. Covering of the water surface
3. Reduction of the exposed water surface
4. Use of underground storage rather than the use of surface storage
5. Integrated operation of reservoirs
6. Use of chemicals for retarding the evaporation rate from the reservoir surface.

Out of all these methods, the last method has evoked the maximum response from all over the world, and has been considered to be the only practical solution for conservation of fresh water, inspite of its various limitations and disadvantages in high cost of application in normal conditions. The use of chemicals, called **Water Evapo-Retardants (WERs)**, for controlling the evaporation rate from the surface of reservoirs is therefore, discussed here in details.

A non toxic chemical, capable of forming a thin *monomolecular film* over the water surface, is generally spread over the reservoir water surface in powder, liquid or emulsion form. *The resulting film prevents energy inputs from the atmosphere, thus reducing evaporation. Such a film, however, allows the passage of enough air through it, to avoid any harmful effects on the aquatic life due to shortage of oxygen.*

Fatty alcohols of different grades like : Cetyl alcohol ($C_{16}.H_{33}.OH$) popularly called *hexa decanol*, Stearyl alcohol ($C_{18}.H_{37}.OH$) popularly called *Octadecanol*, and Behenyl alcohol ($C_{22}.H_{45}.OH$) called *docosanol*, or a mixture of these chemicals, have been generally used and found to be quite suitable. These chemicals should, however, be 99% pure for getting the desired properties of *monolayer*. National Chemical Laboratory, Pune, has developed one more compound by synthesising alkoxy ethanols.

In general, all such chemical compounds should possess the *following properties* :

- (i) the chemical compound (WER) should be tasteless, odourless, non-toxic, non-inflammable, and should not produce any effect on the quality of water.
- (ii) the chemical should easily spread and form an even compact cohesive and efficient monomolecular film on the water surface.
- (iii) the thin film formed by the chemical should be pervious to oxygen and carbon dioxide, but tight enough to prevent escape of water molecules.
- (iv) the thin film formed by the chemical should be durable, and should be able to re-seal itself, when broken due to external disturbances such as wind, waves, etc.
- (v) the chemical and the film formed by it should not be adversely affected by the water borne bacteria, proteins and other impurities present in the water body.

The use of chemical WERs has, however not been found to be cost effective for mass scale use, and has further not been found to be suitable under the following conditions :

- (a) when the wind velocities exceed 10 km/hr or so.
- (b) when the temperature rises above 40°C or so.
- (c) when the size of the water body is relatively large.

Development of cheaper WERs capable of withstanding higher wind speeds upto about 20 km/hr and having strong cohesive forces and properties of self spreading and re-uniting to maintain the monolayer in resilient state even at high wind velocities, is therefore of vital importance. Moreover, the life of the film formed, must be longer, so as to reduce the frequency of application to about 3 to 7 days from its present frequency of 24 hours. Development of such chemical WERs is the subject matter of present research.

Other long term evaporation control measures like *plantation of trees to act as wind breakers**, *reduction of exposed water surface by covers*, *underground storage of water*, *integrated operation of reservoirs*, etc. have been employed in some parts of the country. The effectiveness and economics of these methods are, however, yet to be established.

In India, the water conservation methods are presently being adopted only in draught prone and scarcity areas, since large scale use of such methods on all the reservoirs of the country is not found to be economical or practically unfeasible due to their large size and adverse meteorological factors.

18.17.2. Absorption Losses. These losses do not play any significant role in planning, since their amount, though sometimes large in the beginning, falls considerably as the pores get saturated. They certainly depend upon the type of soil forming the reservoir.

18.17.3. Percolation Losses or Reservoir Leakage. For most of the reservoirs, the banks are permeable but the permeability is so low that the leakage is of no

* Trees having lesser evapotranspiration should only be chosen and identified.

importance. But in certain particular cases, when the walls of the reservoir are made of badly fractured rocks or having continuous seams of porous strata, serious leakage may occur. Sometimes, pressure grouting may have to be used to seal the fractured rocks. The cost of grouting has to be accounted in the economic studies of the project, if the leakage is large.

18.18. Reservoir Clearance

The removal of trees, bushes and other vegetation from the reservoir area is known as reservoir clearance. It is an expensive operation and difficult to be justified on cost-benefit considerations. Non-clearance of such vegetation may lead to the following troubles :

(i) decay of organic material may create undesirable odours and tastes, and hence, becomes important for water supply reservoirs.

(ii) trees projecting above the water surface may create undesirable appearance, when the reservoir is to be used for recreation and tourism purposes.

(iii) bushes, trees, etc. will float and may create debris problems at the dam.

18.19. Selection of a Suitable Site for a Reservoir

It is almost impossible to select a perfect ideal reservoir site. But its selection is guided by the following factors :

(i) A suitable dam site is available. The cost of the dam is generally a controlling factor in the selection of a reservoir site.

(ii) The geological formations for the reservoir banks, walls, etc. should be such as to entail minimum leakage.

(iii) The geology of the catchment area should be such as to entail minimum water losses through absorption and percolation.

(iv) The site should be such that a deep reservoir is formed. A deep reservoir is preferred to a shallow one, because of lower land cost per unit of capacity, less evaporation loss, and less possibility of weed growth.

(v) The reservoir site must have adequate capacity.

(vi) Too much silt laden tributaries should be avoided as far as possible.

(vii) The reservoir basin should have a deep narrow opening in the valley, so that the length of the dam is minimum.

18.20. Reservoir Induced Seismicity

During the recent times, a very strange phenomenon has been observed in several dams of the world. What actually has happened is that the reservoir basins or areas which were seismologically inactive, started showing seismic activities, as the reservoir was filled up with water. The magnitude of these earthquakes is found increasing with the filling of the reservoir, giving severe shocks when the reservoir is full. In all such cases, the epicentre is always seen to be located within the reservoir or along its border.

An important Indian example of such an earthquake has been in the case of Koyna dam (Maharashtra), which is situated in the seismologically inactive zone in Peninsular India. However, when the dam got ready and water started collecting (1962) in the reservoir, the earthquakes also started occurring (1963). The frequency and intensity of these earthquakes had gone on increasing, as more and more water went on collecting in the reservoir. On 10th December, 1967, the severest shock occurred with a magnitude of about 6.5, followed by three more shocks of decreasing magnitudes. The epicentres of all these shocks were traced to be within the reservoir area.

Some other examples of such earthquake sites, noticed outside India, are at *Lake Mead* (U.S.A.) ; *Grand Val Lake* (France) ; *Vorgorno Lake* (Switzerland) ; etc.

This phenomenon has been studied by various seismologists, and it has been suggested by some that in most such cases, *there must have existed some inactive faults in the reservoir basin area, which became active again due to the extra-ordinary load of the reservoir water*, thus causing displacements along these faults and consequently resulting in earthquakes. Some other seismologists have suggested that these earthquakes are caused due to *increased pore pressure* in the adjoining rocks, which lowers their shearing strength, resulting in the release of tectonic strain.

Based on these explanations, the various preventive methods suggested for preventing or reducing such earthquakes, include :

- (i) *filling of the reservoir to a limited safe level ;*
- (ii) *reducing pore pressure by draining out water from weaker adjoining rocks ; and*
- (iii) *to actively explore the dam site for the absence of inactive faults before selecting the same.*

Such earthquakes, however, show a decreasing tendency with time. This phenomenon is indeed very complex and interesting, and still needs further research.

18.21. Economic Height of a Dam

The economic height of a dam is that height of the dam, corresponding to which, the cost of the dam per unit of storage is minimum. For this purpose, the estimates are prepared for construction costs, for several heights of the dam, somewhat above and below the level at which the elevation-storage curve shows a fairly high rate of increase of storage per unit rise of elevation, keeping the length of the dam moderate. The construction cost is found to increase with the dam height, as shown in Fig. 18.29.

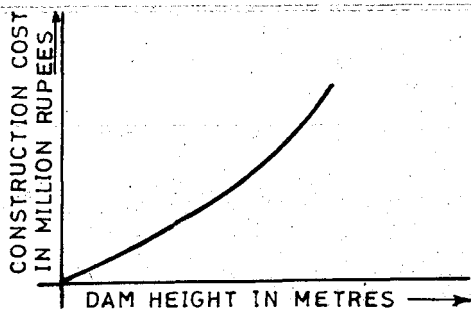


Fig. 18.29

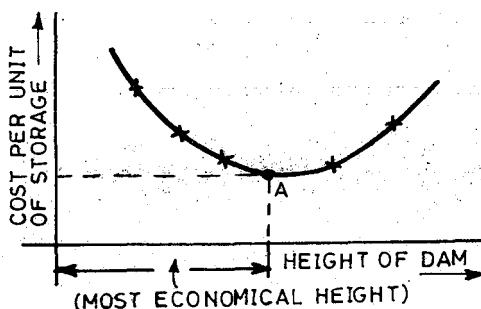


Fig. 18.30

For each dam height, the reservoir storage is known from the reservoir-capacity curve. The construction cost per unit of storage for all the possible dam heights can then be worked out and plotted, as shown in Fig. 18.30.

The lowest point A on this curve, gives the dam height for which the cost per unit of storage is minimum, and hence, most economical.

Example 18.20. *The construction costs for certain possible heights of a dam at a given site have been estimated and are tabulated in the table below. The storage capacity for all these dam heights are also given.*

S.No.	Height of the dam in metres	Construction cost in million Rs.	Storage in million cubic metres
(1)	(2)	(3)	(4)
1	10	4	50
2	20	8	110
3	30	12	180
4	40	18	250
5	50	27	350
6	60	39	500
7	65	50	600

Determine the most economical height of the dam from purely construction point of view.

Solution. The given table is extended, so as to work out the cost per million cubic metre of storage, as shown in col. (5) of Table 18.36.

Table 18.36

S.No.	Height of the dam in metres	Construction cost in M.Rs.	Storage in M.m ³	Cost per unit of storage = $\frac{\text{Col. 3}}{\text{Col. 4}}$
(1)	(2)	(3)	(4)	(5)
1	10	4	50	0.080
2	20	8	110	0.073
3	30	12	180	0.067
4	40	18	250	0.072
5	50	27	350	0.077
6	60	39	500	0.078
7	65	50	600	0.083

The cost per unit of storage is plotted against the height of the dam, as shown in Fig. 18.31. The most economical height is the lowest point of this curve, and it works out to be 30 metres. **Ans.**

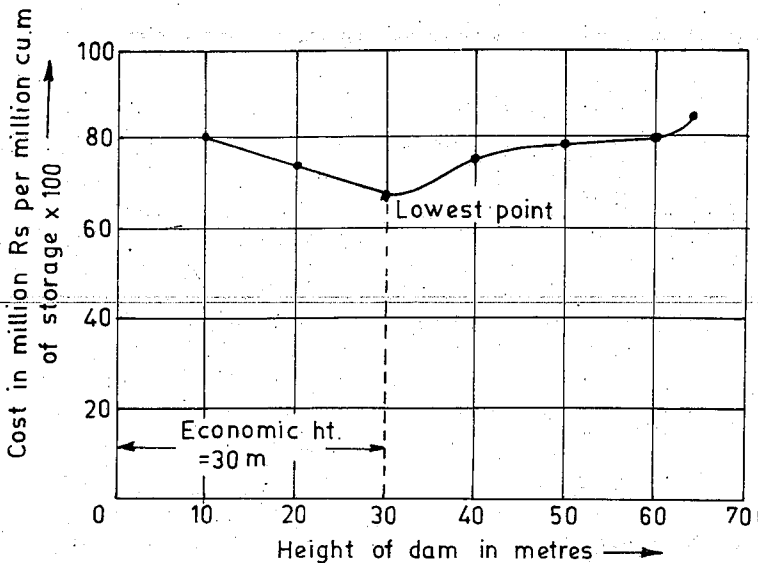


Fig. 18.31.

PROBLEMS

1. (a) What is meant by a 'Reservoir'? Discuss briefly the different types of reservoirs and the purpose served by each type.

(b) Describe briefly the techniques that are employed for computing the storage capacity of a reservoir for different water surface elevations.

2. (a) What is meant by a 'Flood control reservoir'; and what are their different types?

(b) Discuss with a neat sketch, the various storage zones of the dam reservoir.

(c) What factors you will keep in mind while selecting a suitable site for a dam reservoir?

3. (a) Differentiate clearly between the following:

(i) A flood control reservoir and a multipurpose reservoir.

(ii) A retarding basin and a storage reservoir.

(iii) Firm yield, design yield, and secondary yield of a reservoir.

(b) Briefly describe as to how you would fix the storage capacity of a reservoir and the height of the dam required for this storage.

(Madras University, 1976)

4. (a) What is the relation between 'reservoir capacity' and 'reservoir yield'?

(b) How would you fix the capacity of a dam reservoir at a particular river site, provided the inflow pattern and demand pattern are known. Explain the mass curve method which is used for this purpose.

5. (a) Explain how the storage capacity of a reservoir is fixed.

(Madras University, 1973, 1974)

(b) Explain the mass curve method that can be used for determining:

(i) Reservoir capacity for fulfilling given demand.

(ii) Demand rate from a reservoir of a given capacity.

6. Discuss briefly and with necessary neat sketches, the demand patterns for the following types of reservoirs:

(i) Single purpose conservation reservoir.

(ii) Single purpose flood control reservoir.

(iii) Multipurpose reservoir.

7. (a) What is meant by 'flood routing through reservoirs'?

(b) Describe step by step procedure that you will adopt for flood routing computations required for reservoirs under 'trial and error method'.

8. Describe a method for routing flood water through a deep reservoir using the fundamental relation between inflow, outflow (discharge) and storage. Take $Q = C.L. H^{3/2}$ for the spillway.

(U.P.S.C., Engg. Services, 1974)

9. The initial inflow in a reservoir and outflow over the spillway was $30 \text{ m}^3/\text{sec}$. During a storm, the following inflow rates were noted at the ends of successive half day periods: 240, 300, 240, 90, 30 and $30 \text{ m}^3/\text{sec}$. The relationship between discharge over the spillway (Q), and storage rate in the reservoir ($\frac{S}{T}$) may be expressed by the equation

$$Q = \frac{1}{2} \cdot \frac{S}{T}$$

Assuming that the average inflow rate during each half day period is equal to the average of the rates occurring at the beginning and at the end of the period, find the outflow at the end of half hour period.

Hence plot to a suitable scale the inflow and outflow hydrographs.

10. What are the factors on which the rate of silting of an impounding reservoir depends? What is trap efficiency?

Discuss the principal measures that should be undertaken to control the inflow sediment to an impounding reservoir.

An impounding reservoir had original storage capacity for 738 ha-m. The drainage area of the reservoir is 80 sq. km, from which, annual sediment discharges into the reservoir at the rate 0.1153 ha-m

per sq. km. of the drainage area. Assuming the trap efficiency as 80 per cent, find the annual capacity loss of the reservoir in per cent per year.

(U.P.S.C., Engg. Services, 1969)

[Ans. 10%]

11. Write short notes on any four of the following :

- (i) Reservoir losses
- (ii) Reservoir clearance
- (iii) Economic height of a dam
- (iv) Cost benefit considerations in planning dam reservoirs.

[Note. Please See Chapter 20 of "Hydrology and Water Resources Engineering"]

- (v) Reservoir sedimentation and its control
- (vi) Density currents
- (vii) Trap efficiency
- (viii) Estimating the life of a reservoir.