HYDROLOGICAL DIMENSIONING AND OPERATION OF RESERVOIRS

Practical Design Concepts and Principles

by

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Chapter 3

PLANNING FOR DAMS AND RESERVOIRS: HYDROLOGIC DESIGN ELEMENTS AND OPERATIONAL CHARACTERISTICS OF STORAGE RESERVOIRS

In any region where a new storage reservoir is being proposed, the area must be carefully surveyed to establish suitable sites for the construction of the dam. Each alternative site will then receive a detailed assessment to determine the size of dam that can be constructed and the corresponding storage-area-elevation relationships. With a choice of dam sites and various reservoir sizes, the hydrologist or analyst must now assess the water yield from each site and the magnitude and frequency of extreme floods and low flows. This information will facilitate the selection of the best site to satisfy the water demands of the region, as well as provide flow criteria for the design of the outflow structures and the assessment of compensation water releases required for low flow conditions. In addition, where other reservoirs already exist within the river system/network, an assessment is required of the effect of the new reservoir on inflows and outflows of these existing reservoirs. Invariably, the science of hydrology plays crucial roles throughout these exercises.

Hydrology is the basic science underlying water resources management since water cannot be managed without knowing the quantity, areal/spatial distribution, and the flow dynamics or the timing of the movements of water. One of the most important aspects in the practical application of hydrology to water resources management issues is the computation of basic elements of the hydrological regime – including characteristics of streamflow, precipitation, evaporation, dynamics of the water masses, sediment transport and discharge, water quality, etc. – all of which are essential for the effective design, construction, and operation of water resources projects and hydraulic structures. Safety, costs, and efficiency of projects and structures greatly depend on the reliability of hydrological estimates. But reliable determination of hydrological estimates tends to be complicated – in view of the need to account for possible time series variations for different structures with variable project lives. In any case, hydrometeorological data obtainable from networks form the basis for hydrological computations for most water resources projects. Thus, the quality of such computations depends on the availability of hydrological and meteorological data, their resolution in time and space, as well as their accuracy. Indeed, there are several hydrologic estimates and/or procedural steps of a general nature involved in the planning and design of reservoir projects; UNESCO (1982) presents and discusses such general steps, together with the necessary complementary steps that may be specified in relation to specific projects such as flood control, hydropower and conservation storage. This chapter provides a broad overview of the principal hydrologic and related elements that are important to the design and operation of reservoirs to meet specific project goals.

3.1. The Reservoir Design Problem

Several variables require assessment in the design of reservoirs – the number of variables depending on the type of reservoir. Indeed, the examination of a possible site for the construction of a dam and reservoir requires investigations of a variety of issues. Initially, efforts are coordinated to determine whether construction of a dam and appurtenant features is both technically and economically feasible by giving consideration to the geology of the dam and reservoir sites; the kinds and locations of materials available for construction of the dam; and economic, hydrologic, human and geographical factors. Thorough investigation by boreholes, and geophysical exploratory surveys of the types and distribution of material below the land surface at site(s) of the proposed dam(s) and reservoir(s) is a necessary prerequisite to the design and construction phases of the dam project. Also, in the reservoir planning process, it becomes necessary to conduct a mapping and classification of vegetation of the locale, and also an evaluation of environmental impacts, together with the preparation of multidimensional models of evolution of vegetation. Furthermore, in some cases, it is important to take the problems related to site archaeology and paleontology into account. All such works need to be planned well ahead so that there is no interference with construction phase of the program implementation.

Every reservoir would, in general, be characterized by a number of important and unique relationships – namely the volume-elevation, area-elevation, and elevation-discharge relationships. The knowledge of surface area is important in estimating evaporation loss from the water surface. The elevation of the water surface and the uncontrolled discharge of water from the reservoir are dependent on the spillway structure. In addition to the uncontrolled discharge characteristics, a reservoir will have controlled discharge rules, depending on its function. In the design of new reservoirs, a primary goal is to select the best combination of physical characteristics.

The principal basic data needed for reservoir design studies include adequate topographic maps and hydrological records. Records of streamflow are essential for determining the amount of water available for conservation purposes. Historical streamflow data is better supplemented with stochastically generated flow to provide a more reliable design. Rainfall records are also used to supplement streamflow records, or as a basis for computing streamflow where there are no flow records. Although structural-design studies and social problems arising from population adjustments in a locality are not directly related to hydrology, these should, nevertheless, complement the overall planning scheme. It is also noteworthy that, every reservoir that impounds water behind a dam is a real or potential threat to those living downstream of such structure; also, in some locations where earthquake shocks, movements along bedrock faults beneath dams, or collapse of large volumes of earth materials into the reservoir are distinct possibilities, even the most skilled design and continued maintenance may not preclude failures that could be disastrous to life and property. Where the possibility of massive failure in slopes of reservoirs exist – whether empty or full – appropriate steps should be taken to stabilize the slopes; the particular remedy employed depends on the nature of local conditions. Furthermore, in the case of riverflow regulation, the natural balance between rivers and their floodplains can be disturbed; thus, it is necessary to elaborate a comprehensive plan of action in relation to management of the catchment area. In fact, as a result of flow regulation, the heterogeneous meandering river may at times change into a homogeneous straight channel with less habitat diversity, increased transport of sediments, and reduced self-purification. Such potential problems should be anticipated, and feasible corrective measures incorporated into the overall design scheme.

In general, reservoir size selection usually follows after site selection, but the two may be interdependent. The factors pertinent to size selection would normally include the project function(s), the physical factors, economic factors, environmental considerations as well as social considerations. For economic reasons, it is important that the required water-storage capacity be adequate but no more than necessary. In any case, reservoir operation usually offers significant flexibility – so that optimal conditions can be achieved by making appropriate post-construction adjustments to the reservoir system.

3.1.1. RESERVOIR-SITE SELECTION

The choice of a dam site is governed by the purpose(s) for which the water resource(s) is to be developed; the physical suitability of available sites to serve those purposes safely and economically; and the necessary permission or government authorization to use the particular location of choice. In any case, no two river basins are completely alike – differing not only from the engineering viewpoint but also in cultural, social and economic background and are, in some cases, affected by local, national or even international political issues. Thus, apart from being faced with engineering problems, river basin development can involve several issues and considerations that are entirely non-technical in nature – and which factors may in fact significantly influence the siting of a project.

Morphological, geological, hydrological, topographical and terrain conditions, as well as technical and manpower availability, are among the most important factors affecting reservoir-site selection. The ideal morphology for a reservoir site would be a narrow valley-gorge for the dam, with a flat, broad valley for the reservoir on its upstream side. In practice, however, sites of such nature are rare; but since design work is a steady quest for the optimum compromise among conflicting factors, the morphological aspects would play a very important role in the overall reservoir design exercise. Geology is also of decisive importance in reservoir design; in dam site selection, it is advantageous and also profitable to look for the highest possible elevation of solid rock, thus reducing excavation work. Additional geological considerations require that the site not be on formations that leak excessively, and that there not be any risk of large landslides, rockslides, or rockfalls into the reservoir. The role of hydrology in the reservoir design exercise pertains to the determination of the design flood discharge for spillway and diversion structures, as well as determination of capacity (to include seepage/infiltration, wind-caused wave effects, evaporation and sedimentation issues) and guaranteed water yields.

Ultimately, to obtain economical storage capacity, a reservoir site should be wide in comparison to the dam site and should be on a stream having a low or gentle gradient in order to obtain a long reservoir in proportion to the height of the dam. In general, the location and physical description of the project area(s), together with alternative layouts should be carefully evaluated. There is the need to conduct a major program of subsurface investigation drilling activities, soil and rock sampling and analyses, and geophysical surveys at the site or area in order to generate adequate geological data for the conceptual design studies. Other considerations may consist of population surveys/studies of the project area, community resettlement programs, agricultural development plans, etc.

3.1.2. THE CASE FOR MULTIPURPOSE RESERVOIRS

The design of multipurpose reservoirs present complications when quantities of water released at any time may or may not be made to serve more than one purpose (see, illustrative example in Figure 3.1). Indeed, multipurpose reservoirs are generally called upon to satisfy diverging requirements in accordance with the variety of objectives served.

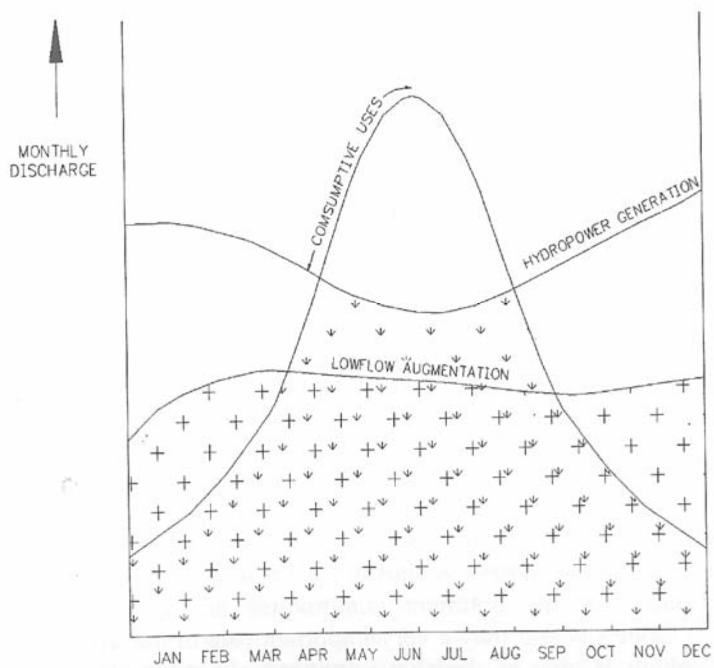

Figure 3.1. Scheduling water releases for different purposes (adapted from Buras, 1972 and Hall, 1964)

Planning for a single purpose in a single reservoir analysis is the simplest task because there tends to be no conflicting demands to be evaluated. Multipurpose reservoir analysis, on the other hand, generally requires detailed sequential analysis to avoid conflicts among purposes. The selection of time interval for analysis depends on the variability or randomness of the supply and demand. Priorities among purposes must be established during the analysis; flood control usually has the highest priority.

In fact, the conservation storage in a reservoir may provide some flood protection – albeit its primary purpose is to provide water whenever demand are in excess of supply, and to provide head for hydroelectric energy. The upper and lower limits of conservation storage may vary seasonally. If several conservation purposes exist, then priorities will have to be established to ensure that the highest priority demands are met. The conservation storage can be subdivided by allocating a portion of the total storage to a buffer zone; when water in storage gets down to the buffer zone, only the highest priorities would be supplied, or supplies for all purposes will be reduced.

The simulation strategy for a multipurpose reservoir can be performed using period-of-record flow data or by using isolated critical flow periods. For flood control analysis, the time interval is generally relatively small and local events are usually used. For most conservation purposes, a monthly time interval is sufficient to evaluate the reservoir operation in planning studies. In general, shorter sequenced critical periods could be used to develop plans or screen alternatives; then, with just a few alternatives, a longer sequence should be used to evaluate performance over a range of hydrologic conditions.

Besides the unquestionable advantages, one major problem with multipurpose storages is the need for coordinating a variety – sometimes conflicting – interests; for instance the conflicting demands of flood control and other uses for storage space tends to create substantial difficulties. In most practical situations, these goals need to be optimized. Accordingly, in planning for multipurpose reservoirs, the site and location of the dam, the capacity of the reservoir, and the allocation of costs must be so-determined as to reconcile – as far as possible – the conflicting interests.

3.2. Multireservoir System Layout and Analyses

A multiple reservoir decision arises when there are two or more reservoirs operating for a common purpose. Although reservoir releases in a multireservoir system can be established on an individual reservoir basis, increased benefits can generally be realized by using a systems approach to determine the releases.

All reservoir systems can be characterized as combinations of tandem (in series) and parallel reservoirs. For *tandem reservoirs*, the release to meet downstream demands is made from the downstream reservoir while releases are made from the upper reservoir to meet local needs and to keep all the reservoirs at some predetermined balance; one exception to this would be when the upper reservoir is a hydropower project. Ideally, the upper hydropower project should only release for power generation while the lower reservoir re-regulates the flow for downstream requirements. The analysis of reservoirs in series is limited by problems of serial correlation between reservoir inflows that are analogous to the serial correlation of inflow between periods. For *parallel reservoirs* operating for a common downstream point, any combination of releases that provides the required total release would be satisfactory to the downstream user; the question to be answered is how much to release from each. In fact, when parallel reservoirs serve the same purposes, the apportionment of required releases among the reservoirs becomes a major decision.

For a system of reservoirs within a river basin, reservoirs may be considered as either acting independently or acting jointly. A reservoir is said to be acting *independently* when the damages and/or shortages to water excesses (or floods), and water shortages (or droughts) can be identified with releases made from that reservoir.

An independent reservoir may be said to be a storage reservoir acting in isolation, i.e., a storage facility operated independently of any other reservoir(s). For instance, referring to Figure 3.2, reservoirs 1 and 4 may be identified as acting independently because all damages due to floods or water shortages between reservoirs 1 and 2 are ascribed to reservoir 1 and all losses/damages immediately downstream of reservoir 4 are attributed to reservoir 4. A set of reservoirs is said to be acting *jointly* when the benefits and damages from them can be identified only with the joint operation of the set as a whole and cannot necessarily be attributed to the individual reservoirs in the set as such. Thus, a dependent reservoir is a cooperating one, and whose design and operation are dependent upon other reservoirs. Referring to Figure 3.2 again, reservoirs 2 and 3 may be classified as acting jointly since the damages/losses below the confluence of the two streams and the damages upstream of reservoir 3 cannot be identified separately with reservoir 2 or 3. The cooperating effect or dependency of reservoirs may arise if, for instance, reservoir inflow depends partly or wholly on the regulated outflow of upstream reservoirs; reservoir outflow is imposed by joint operation with downstream reservoirs; or reservoir outflow is affected by water released from reservoirs in adjacent river basins.

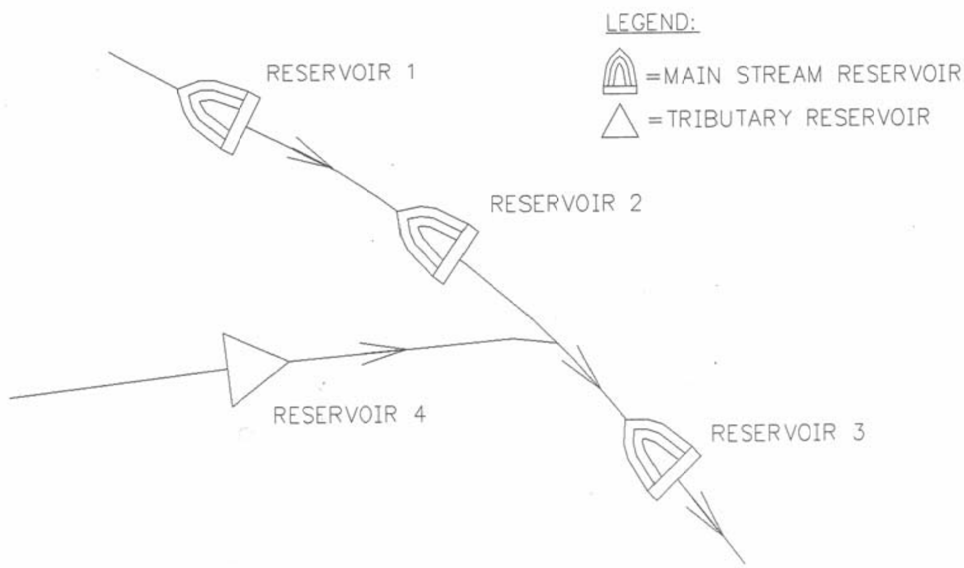

Figure 3.2. A schematic of reservoirs acting independently and jointly for a hypothetical river basin

Figures 3.3 show a hypothetical but representative and typical river basin system – in which reservoirs may be placed in parallel and in series, and in which reservoirs may be acting jointly or independently.

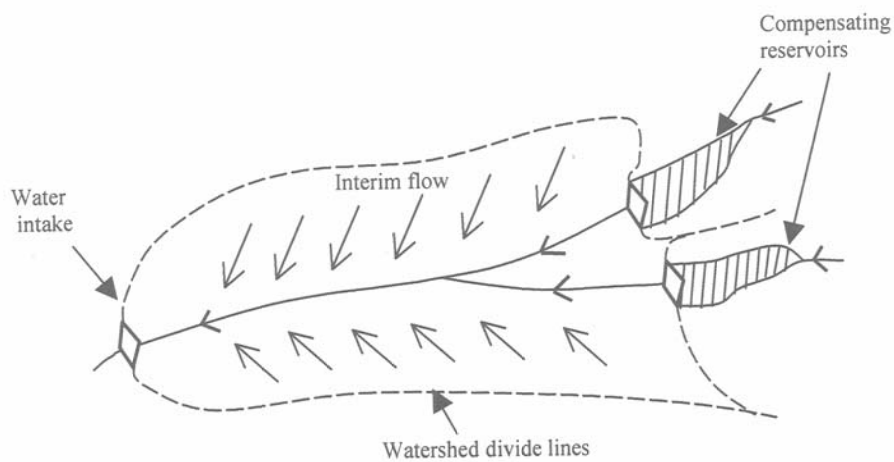

Figure 3.3. Illustrative sketch for typical network of compensating reservoirs

3.3. Hydrological Basis for the Determination of Reservoir Storage Capacity

The primary purpose of storage reservoirs is to provide a means of surface water regulation, both with respect to time and amount; the reservoir storage capacity, together with its operating policy, provides an indication of the extent to which streamflow can be stored for later release or other uses.

Although physically inseparable in some cases, distinction is often made between the various reservoir storage components. In general, reservoir storage capacity can be divided among three major uses (Figure 3.4), namely:

1. The *active storage*, used for streamflow regulation and for water supply;
2. The *dead storage*, required for sediment collection, recreational development, and/or hydropower production; and
3. The *flood storage capacity*, reserved to reduce potential downstream flood damages.

Often these components of reservoir capacity can be modeled separately, and then the total reservoir capacity, K, calculated as the sum of the dead storage capacity (K_d), the active storage capacity (K_a), and the flood storage capacity (K_f). However, since the required active storage capacity varies throughout the year, and since flood storage may not be required in all seasons, a less conservative estimate of the total capacity (K) will be the sum of dead storage and the maximum required active storage and flood storage in the flood season *or* the maximum required active storage in non-flood seasons, whichever is greater. Thus, with an initial storage volume of S(t) at the beginning of period *t*, the following relationships hold true (Loucks *et al.*, 1981):

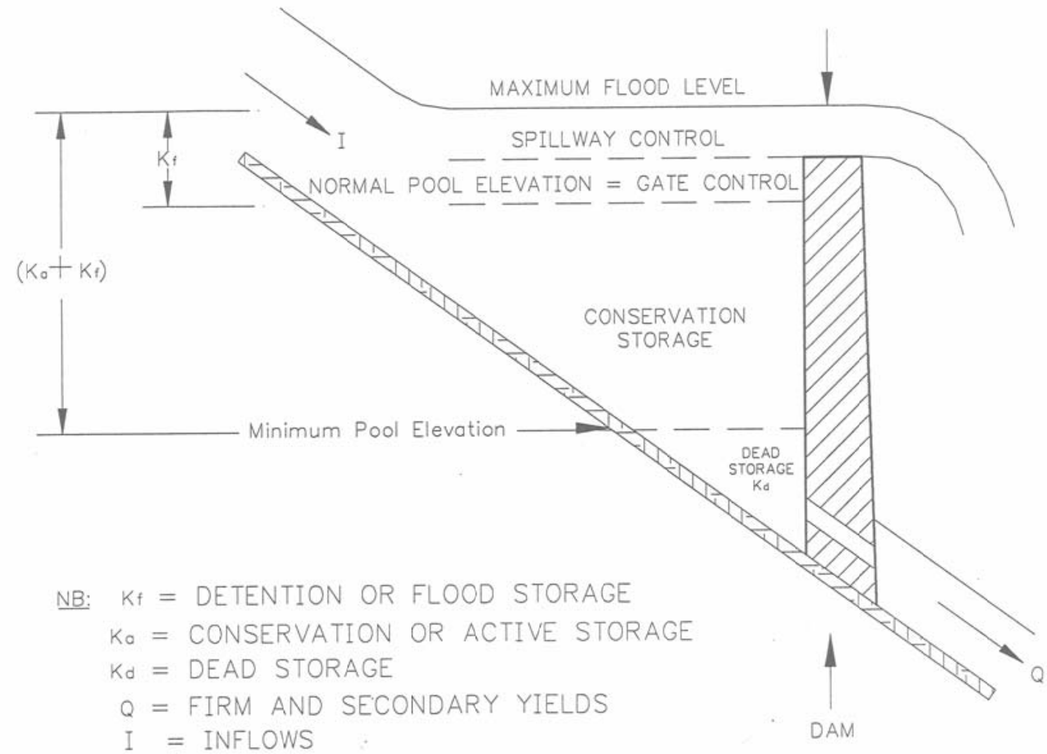

Figure 3.4. Reservoir storage zones in multipurpose reservoirs

$$K \geq K_d + S(t) + K_f \qquad \forall t \text{ in Flood Season} \tag{3.1}$$

$$K \geq K_d + S(t) \qquad \forall t \text{ in Non-Flood Season} \tag{3.2}$$

This definition of total capacity permits the trade-off between active and flood storage capacity when and if there can be a trade-off in different seasons (Loucks *et al.*, 1981). Indeed, since reservoirs are designed to reduce flood damages and also to meet low-flow augmentation demands, one must simultaneously take into consideration the drought period inflows and the flood year inflows in any attempts to find optimal capacities for the reservoirs; this helps to side-step the problem of over-design which could result by computing the total capacity via the conservative approach which is based on the 'blind totals.' Although, strictly speaking, the flood attenuating effect of multipurpose reservoirs cannot be enhanced unless the reliability of supply for other purposes is sacrificed, a balance can nevertheless be drawn among all the uses in an optimal design scheduling. It is also noteworthy that, the storage capacity reserved for sedimentation may be utilized for a considerable period after the construction of the dam; thus, in some cases, the water in the dead storage below the sedimentation level may also be utilized in abnormally dry seasons.

The analysis of all possible alternative configurations in the hydrological dimensioning of multiple reservoir systems is indeed a formidable task. In general, any site-/spatial location-based advantage of one reservoir over the others should be very carefully evaluated, and 'priority' given to this factor for utilization in the design process. Ultimately, the project should be economically justifiable, and should be formulated so as to maximize net economic benefits whiles providing a reasonable guarantee of dependable water demands from the reservoirs, and also concurrently satisfying the necessary flood protection, etc.

3.3.1. A MODEL OF CAPACITY ALLOCATION AND SURVEY OF WATER DEMANDS IN MULTIPURPOSE RESERVOIRS

The management of an integrated or multipurpose water control system presents a different kind of problem to that of a single purpose one. A typical allocation of storage in a multipurpose reservoir includes flood control storage, conservation storage, hydroelectric power storage, sediment storage, and a buffer storage. In fact, water resources are controlled and regulated to achieve a wide variety of objectives – and a single purpose reservoir may be considered as a special case of a multipurpose one.

A model diagram of capacity allocation in a typical/representative multipurpose reservoir is provided in Figure 3.5; Figure 3.5(a) shows the case when flood control requirements are less severe, whereas Figure 3.5(b) depicts situations where the requirements for flood control are rather critical. Although sometimes physically inseparable, distinction is usually made between the different components of reservoir storage. In each case, the determination of storage requirements is based on somewhat different hydrologic data. A brief general survey of the demands of some different water uses and capacity allocations, together with a discussion of the hydrological basis for the computation/estimation of storage requirements is presented below for the rather pertinent and commonly designed-for uses.

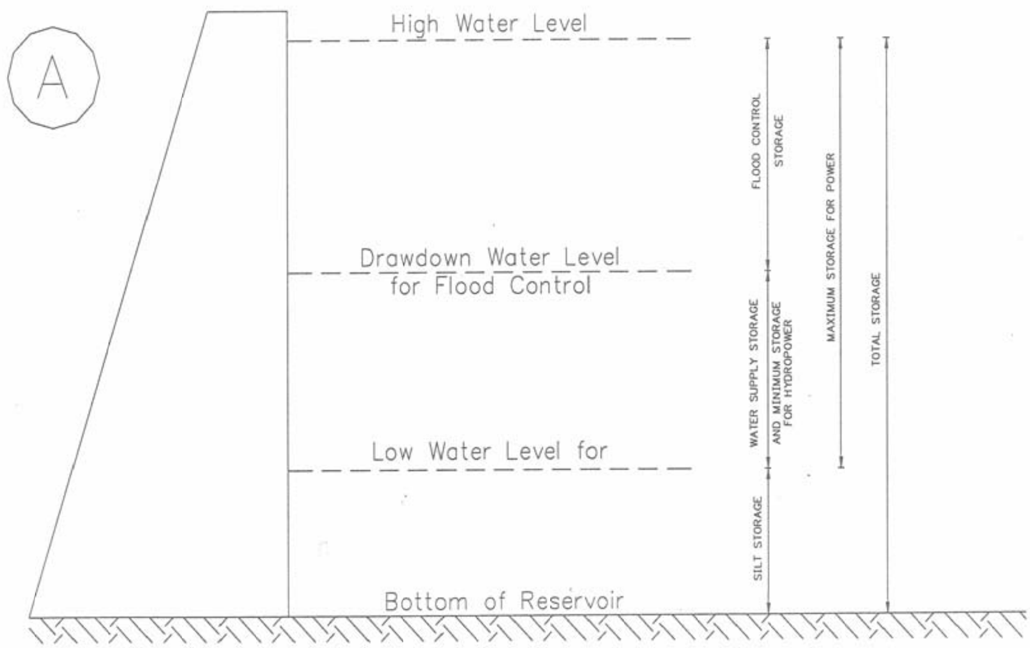

Figure 3.5(a). Allocation of reservoir storage capacity (adapted from ECAFE, 1962)

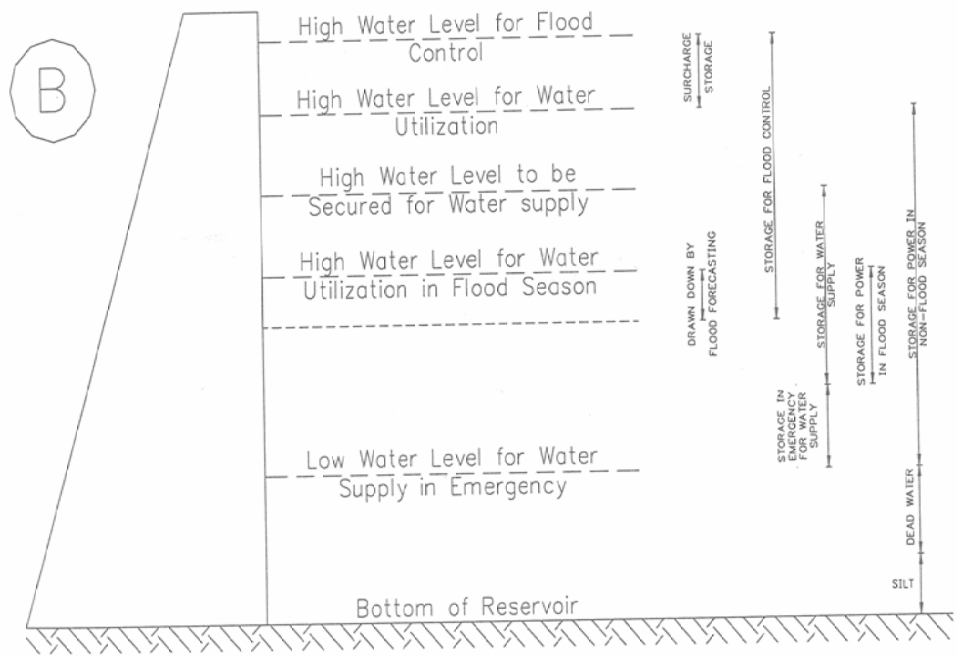

Figure 3.5(b). Allocation of reservoir storage capacity (adapted from ECAFE, 1962)

3.3.2. ESTIMATING THE ACTIVE STORAGE NECESSARY FOR FLOW REGULATION AND WATER SUPPLY

The determination of the reservoir storage requirements for all conservation purposes other than hydropower is considered under this topic. Several authors (e.g., UNESCO, 1982; HEC, 1977, 1975; Sokolov and Chapman, 1974; Davis and Sorensen, 1969) elaborate specific steps to be considered in estimating reservoir storage requirements for water supply and irrigation.

In general, requirements for public water usage and industrial demands can be estimated based on population forecasts and assessments, as well as industrial surveys. Future requirements due to growing population and anticipated industrial expansion should also be taken into consideration at the initial planning stages. Additionally, it must be recognized that an improved standard of living of a population usually means a further increase in water requirements for domestic water supply.

The irrigation water demand is almost always periodic in nature. Moreover, there are considerable differences in water quantities required during the irrigation periods of individual years, since the demand is dependent on the distribution of rainfall, among other hydro-meteorological factors. Furthermore, the growing seasons of various species of crops are different – and so are the beginning and duration of the irrigation seasons. In general, the quantity of irrigation water required depends on a number of climatic factors – namely, the volume and distribution in time of rainfall, temperature, humidity, wind conditions and related factors; the physical and chemical properties of the soil and subsoil; and the water demand of the specific crops being cultivated. These factors determine not only the volume, but also the distribution in time of irrigation water. Under similar climatic and soil/pedological conditions, the distribution of crop water requirements depends only on the kind of crops grown – and even that may differ considerably. In extensive irrigation schemes and systems, demand fluctuations can be curbed by a suitable combination of crops and areas of cultivation, under a mixed cropping program, leading to an almost uniform water demand vis-à-vis the supply.

Another important component of conservation storage relates to demands for low flow augmentation for navigational and recreational activities, as well as for water quality control. A navigation development must offer long-range dependability in order to attract shippers and promote growth of terminals along the waterway. Also, the navigation channel must be dependable in terms of depth and of the range of fluctuation of water levels, and should also be reasonably free of wide variations in streamflow and the corresponding velocity changes. The objective of limiting the variation in velocities and water levels in the reservoirs is attained by careful routing of discharges through the reservoir system, and by imposing constraints on the rate of change of discharge from the dams. In addition, recreational use, pollution abatement, preservation of scenic and wilderness areas, etc. helps to maintain or improve health and welfare of populations. Indeed, pollution reduces the utility of water for municipal, agricultural and industrial uses and seriously diminishes the recreational and aesthetic value of lakes and reservoirs; hence water quality control has become a very important objective of many water projects.

Some of the commonly used methods for estimating active storage requirements include the Mass Diagram analyses (Fair *et al.*, 1966; Rippl, 1883), the Sequent-Peak procedure (Fiering, 1967; Thomas and Burden, 1963), Simulation and Optimization techniques and analyses (Wurbs, 1996; Loucks *et al.*, 1981; Hall and Dracup, 1975); some detailed discussion of these methods are presented in Chapters 7 through 10.

3.3.3. HYDROELECTRIC POWER POTENTIAL OF STORAGE RESERVOIRS

Reservoirs may be created for the provision of power for economic development, improved living standards, and an overall enhanced quality of life. The major hydrologic difference between the routing studies for hydropower and other project purposes is that, the quantity of water required in a given time period is a function of two independent variables – power demand and effective head. Furthermore, the quantity of water required is a direct function of the power demand, but an inverse function of the head.

In general, the production of hydroelectric energy during any period at any particular reservoir site is dependent on a number of factors – including the installed plant capacity; flow through the turbines; average productive storage head; number of hours in the period; plant factor; and a constant for converting the product of flow, head, and plant efficiency to kilowatt-hours of electrical energy. The energy produced in period t, E(t) [in kilowatt-hours, Kwh], is proportional to the product of the plant efficiency, η, the productive storage head, H(t), and the flow q(t) through the turbines, viz.:

$$E(t) \propto [\{\eta\}\cdot\{H(t)\}\cdot\{q(t)\}] \tag{3.3}$$

For the total realization from a hydropower project, leakage through the dam, as well as head losses through the penstock, turbines, and draft tubes must be accounted for. The tailwater elevation is also to be given serious consideration in power studies – since a 1-meter loss in head may represent a major economic loss (UNESCO, 1982). Also, the large fluctuations of releases required to meet peak load demands can cause severe bank erosion – and this may require studies to determine storage requirements of downstream re-regulation dams, and the resulting effects of tailwater elevations.

3.3.4. STORAGE-SPACE FOR FLOOD MITIGATION: THE RESERVOIR FLOOD STORAGE CAPACITY DESIGN

The storage space reserved for the accommodation of floods generally cannot be utilized for other purposes. This reservoir volume must therefore be reserved to retain a flood wave, whenever it may occur. As an exception to this general rule, a two-fold utilization may be considered, in periods only where – under the prevailing climatic conditions – no flood, or more precisely no major floods are likely to occur; thus, two-fold utilization is necessarily periodical and partial, i.e., it can extend only to a part of this space.

The objective of all flood control operations is to regulate damaging floods to minimize potential damages. In compliance with the multipurpose reservoir operating rules, storage space in the reservoirs is reserved – and during flood runoff periods, this is used to reduce downstream flood crests. During periods of substantial flood flows, flood control requirements always have top priority over *all* other objectives of system operation. Although the primary objective of flood control operation is the reduction of flood crests to non-damaging levels, a secondary objective of providing timely flood warnings is also important. When the flood cannot be regulated to prevent all damages, timely flood warning can result in substantial lessening of the flood damages, and may indeed prevent loss of life.

To determine the optimal capacity of a reservoir from the point of view of flood control, the inflow data for the worst flood year, selected from the historical inflow data of a specified duration, would typically be utilized in the ensuing analysis. (It is noteworthy that, from the point of view of low-flow augmentation, the inflow data will be based on the severest drought period on record.) In the design of flood control reservoirs, it is usually assumed that the period between floods is long enough for the reservoir to be emptied before the arrival of the next flood. However, there are climatic conditions – mainly in areas affected by tropical cyclones – where two major floods may occur in succession. Under these conditions the period between the floods cannot be ignored and has to be considered as a design variable. Typically, the year is divided into a number of equal time intervals; the number of subdivisions should be such that salient features of the inflows, such as peak flows and duration of a flood are reflected in the design with a reasonable degree of accuracy. If the number of subdivisions is not large enough, a certain amount of information on the inflow rates may be lost since, within every subinterval, the inflow is averaged out.

Ultimately, the determination of flood control storage for a reservoir project is based on hydrologic analyses that are governed by a set of project formulation criteria. The guiding formulation principles generally would require that the project should be economically justified, and that the project should *not* result in a significantly increased flood hazards for any flood event that exceeds the design capacity. The fundamental considerations in the design exercise are elaborated below.

Analysis and Synthesis of Flood Flows: Floodflow Computations

The probability or likelihood of a flood peak of given magnitude is often described by its *return period* (also called the *recurrence interval*). The *return period*, T, of a flood is the expected number of years before the occurrence of a flood of equal or greater magnitude; the probability that a T-year flood will be exceeded in any given year is 1/T.

If Q_p is the random annual peak flood flow, and Q_{pt} is a particular peak flood flow having a return period of T years, then the probability of Q_p equaling or exceeding Q_{pt} is given by:

$$\Pr\{Q_p \geq Q_{pt}\} = \frac{1}{T} \tag{3.4}$$

This is the *exceedance probability*, and its distribution function for continuous distributions is defined as follows:

$$\Pr\{Q_p \geq Q_{pt}\} = 1 - F_{Qp}(Q_{pt}) \tag{3.5}$$

or

$$F_{Qp}(Q_{pt}) = 1 - \frac{1}{T} \tag{3.6}$$

The expected annual flood damage at a potential flood damage site can be estimated from knowledge of the exceedance probability distribution of peak flood flows and the resulting damage. The peak flow, Q_{pt}, at any potential damage site resulting from a flood of return period, *T,* will be a function of the upstream reservoir flood storage capacity, K_f, and the reservoir operating policy, viz.:

$$Q_{pt} = f_T(K_f) \tag{3.7}$$

Assuming a known operating policy for flood flow releases, this function, $f_T(\ldots)$, can be defined by routing a series of floods through the upstream reservoir that has various flood storage capacities, K_f, and a known operating policy to the downstream potential damage site. This will require a flood routing simulation model (see, e.g., Viessmann *et al.*, 1977). The flood hydrograph or peak discharge that is ultimately adopted as the basis for design of the structure is called the *design flood*; the selection of the appropriate design flood does indeed require a hydro-economics analysis of the problem.

Flood Control by Multipurpose Reservoir Systems

For the preliminary estimation of flood storage capacities in multi-reservoir systems, and that also involves several potential damage sites, a single design flood can be selected. Using this design flood, the system may be simulated to determine the effect that any combination of flood storage capacities has on reducing the flood peak at some downstream potential damage site(s). The difficulty in this exercise is in obtaining a simple expression for this effect as a function of all the upstream reservoir capacities – hence the need to introduce the concept of an *equivalent flood storage capacity* at a fictitious or actual reservoir site just upstream of the potential damage site. The *equivalent flood storage capacity*, $K_{i,f,}$ at reservoir site *i* will be that capacity which reduces the flood peak at the downstream damage site by the same amount as the actual flood storage capacities, $K_{i,f}$, in the upstream reservoirs at sites $i = 1, \ldots\ldots\ldots.n$ {*n* being an integer indicating the reservoir sites}. This principle of 'equivalent flood storage capacities' is demonstrated in a comprehensive manner in Loucks *et al.* (1981). The flood storage capacities, $K_{i,f,}$ may indeed represent conservative estimates of the capacities needed to reduce flood peaks downstream if active storage capacity or dead storage capacity is included in the total reservoir capacity. This is especially so because existing lakes will tend to increase channel widths and some flood storage may be provided by unused active reservoir storage capacity at the time of a flood – both of which conditions result in reduced flood peaks downstream, even without additional flood storage capacity at the reservoir sites.

In general, for flood protection in a multipurpose reservoir, a certain volume of the storage reservoir is kept empty – ready to receive water during the peak of the flood. The stored water is discharged after the flood peak has passed. The reservoir receives all the water in excess of a design flow rate, Q_d – i.e., the peak of the hydrograph is 'skinned off' and is delayed by storage. Thus the flow rate downstream is limited to a predetermined maximum value. The stored volume is discharged when the flow rate downstream falls below Q_d. This reduces the peak discharge rate of the downstream hydrograph but increases the base-length of the hydrograph.

Controlled vs. Uncontrolled Storages

The reservoir storage may be controlled or uncontrolled. The storage above a spillway crest (i.e., without gates) is an uncontrolled storage – and its effect on the outflow is determined in accordance with the principles of reservoir routing; it is noteworthy that, even this type of reservoir has a control facility in the form of gated bottom outlets –

i.e., sluice gates. For a fully controlled storage, the outlet must be gated – and the discharge capacity at zero storage must be equal to or greater than the maximum possible rate of inflow.

A fully controlled storage is the most effective way of reducing flood peaks, since only the volume above the design discharge needs to be stored (area [1] in Figure 3.6); for an uncontrolled storage, volumes corresponding to areas [1] and [2] have to be stored. Furthermore, the controlled storage is emptied in time t_e after the inflow has dropped below Q_d. The shape of the falling part of the Q_c hydrograph depends on the gate and operating characteristics. The emptying time for the uncontrolled storage is measured by the interval between the point of intersection of the I and Q_u hydrographs, and the point where the recession limbs coincide.

Associated with the higher efficiency of controlled storages is some reduction in the overall safety, since gates may fail to open. Hence, an ungated emergency spillway is usually included in controlled storage schemes. Klemes (1973) elaborates a graphical design procedure for the estimation of the controlled storage volume for a given return period and discharge rate, Q_d.

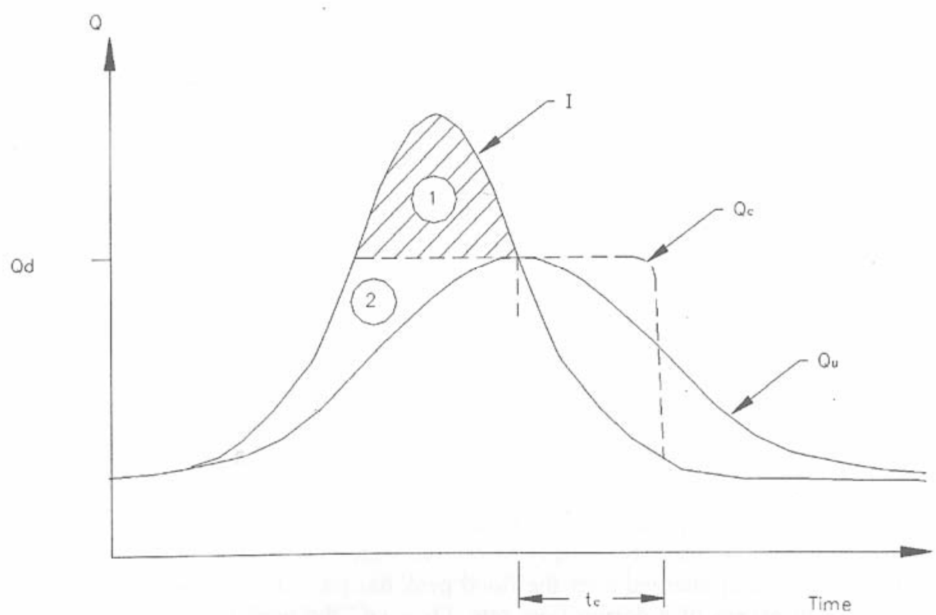

Figure 3.6. Storage for flood control
(hydrographs Q_c, Q_u and I represent the controlled discharge, uncontrolled discharge and inflow, respectively)

3.3.5. SILTATION OF RESERVOIRS AND SEDIMENT RESERVE STORAGE

A major problem associated with the design and operation of reservoirs is siltation. In fact, unless carefully constructed and maintained, the ultimate destiny of all reservoirs is deterioration and failure or filling by sedimentation. If the sediment inflow is large compared with the reservoir capacity, the useful life of the reservoir may be very short. Reservoir planning must therefore, of necessity, include consideration of the probable rate of sedimentation – in order to determine whether the useful life of the proposed reservoir will be sufficient to warrant its construction.

Erosion and sedimentation processes are indeed parts of the geological evolution of landscapes, of which rivers and river processes play a significant role. Also, major engineering works cause a redistribution of water and sediment that affects channel processes. Dams and reservoirs interrupt and alter the natural flow of water and sediment through the drainage system. The reservoir is a sink producing hydraulic conditions that reduce the velocity of flow, often to near zero, within the reservoir. The change in velocity is usually sufficiently rapid as the flow enters the reservoir. The bed load sediment particles settle out first and the suspended sediment particles will travel farthest into the reservoir before settling out (V. Nagy, 1990). Since hydraulic structures on river channels disturb natural processes of riverbed adjustment, any hydraulic project requires the consideration of both the regime of sediment transport and the hydraulic structure's effect on sediment motion.

Ultimately, a reservoir collects sediment, and the eroding capacity of the downstream flow tends to increase compared with that under unchanged conditions. This causes more intensive local erosion adjacent to the dam as well as an intensification of channel processes further downstream (USBR, 1977). These downstream changes may occur over long distances (>10 km); aggradation is intensified upstream in the reservoir backwater, thus producing a rise in water levels.

Problems of sediment transport enter the reservoir design in several ways. The designer has to consider the response of the river to the reservoir upstream and downstream from it, the siltation of the reservoir itself, and the handling of the river during the construction of the reservoir. It is noteworthy that, prediction of the rate of sediment supply to the reservoir and the amount retained in the reservoir is still a highly empirical art.

The Nature and Extent of Reservoir Sedimentation

Soil erosion and sedimentation is a universal problem in the design and management of storage reservoirs. In any case, sediment yield attributable to Asia is estimated to be more than twice the world average. For instance, the maximum sediment load can be as high as 1600 kg/m^3 in the Yellow River and its tributaries and the Yongding River in China – and this has resulted in serious sediment deposition problems in the reservoirs constructed on these rivers.

In fact, in some of the worst cases of reservoir sedimentation on record, more then 50 percent of the storage capacity had been lost after few years of operation. The average annual rate of loss of the storage capacity (i.e., ratio of average annual deposition volume to total storage volume) has been estimated at 2.3 percent (Chun Hong, 1995). Stall and Lee (1980) have reported that in some reservoirs in Illionois (USA), the rate of capacity loss is about 4.8%. Also, the 76 m high Warsak Dam on the Kobul River in Pakistan lost 18% of its storage volume in the very first year of operation. Furthermore, Pleskov (1985) provides detailed characteristics of the sedimentation of the six reservoirs constructed on the Don River in Russia (Table 3.1).

As a further example, the case of the Tiszalök barrage in Hungary with a storage space of 10 x 10^6 m^3 is worth mentioning here. In this case, the measured suspended sediment transport of the Tisza River fluctuates between 1 and 9 million tons annually. The discharge characteristics of the river are: Q_{min} = 56 m^3/s, Q_{mean} = 530 m^3/s, and

Q_{max} = 3620 m^3/s. The flow velocity is around 15 - 25 cm/s in the main channel, and between 0.5 - 10 cm/s in the flood plain. During the first two years of operation, an aggradation of 7 m was observed along certain reaches, and in the first 7-year period, 4.5 x 10^6 m^3 deposition was registered in the reservoir; indeed, it has also been reported that the actual deposition during this period was 5.7 x 10^6 m^3, except that 1.2 x 10^6 of the total volume became stirred up and transported away by flood waves (Rákoczi, 1994).

Table 3.1. Characteristics of the Don River reservoirs

Reservoir characteristic	**Reservoir identification number**					
	1	*2*	*3*	*4*	*5*	*6*
Area of the watershed (km^2)	430	403	130.5	271	540	1113
Average runoff, mill. m^3	13.6	23.5	8.2	17.5	34.0	45.6
Volume of the reservoir, mill. m^3	25.0	26.8	8.5	16.3	1.3	52.0
Total volume of sedimentation 10^6 m^3	988	687	273	436	1000	4000
Time of operation, year	7	7	7	5	7	11
Average sedimentation 10^6 m^3 / year	141	98	39	87	143	364

Prediction of Sediment Yield

A large variety of erosion-sediment yield models are available in the literature – with the commonly used methods for prediction of sediment yield elaborated below.

- **USLE.** The universal soil loss equation (USLE) method (Smith and Wischmeier, 1957) computes the soil loss at a given site as a product of six major factors, as follows:

$$A = R \cdot K \cdot L \cdot S \cdot C \cdot P \tag{3.8}$$

where, A is the soil loss per unit area, R is the rainfall erosivity factor, K is the soil erodibility factor, L is the field length factor, S is the field slope factor, C is the cropping management factor, and P is the conservation practice factor. This equation has been used to estimate sediment yields for the design of small reservoirs (Haanet *et. al.*, 1982). Simons and Sentürk (1992) indicate that the USLE is the most widely used regression model for predicting soil erosion.

- **MUSLE.** The modified universal soil loss equation (MUSLE) was proposed by Williams (1975) by replacing the factor R in the USLE model with a runoff factor, as follows:

$$A = 11.8 \cdot \left(V_Q \cdot Q_P\right)^{0.56} \cdot K \cdot L \cdot S \cdot C \cdot P \qquad (3.9)$$

where V_Q is the volume of the runoff (m^3), Q_P is the peak flow rate for the storm (m^3/s), and the other terms have the same meaning as in the USLE method above. This model actually considers both surface erosion and sediment movement in the catchment.

The factors in equations 3.8 and 3.9 (R, V_Q, Q_P), and accordingly the A value, are random variables and consequently the amount of sediment transported by the rivers is also a random variable. Therefore, for the prediction of sediment transport in the rivers, some empirical probability distribution functions should preferably be utilized (Mosaedi, 1998). Indeed, several variations of the above methods, as well as various other methods for the prediction of sediment yield are available in the literature of soil science, hydrology, and water resources.

Reservoir Trap Efficiency

Reservoir sedimentation rates can be estimated by surveys to determine the rate of sediment accumulation in reservoirs that have been in existence for many years. The actual accumulation of sediment in a reservoir depends on the percent of the inflowing sediments that is retained in a reservoir, defined as its *trap efficiency*; this is a function of the ratio of reservoir capacity to total inflows. In fact, the value of the trap efficiency depends on the sediment fall velocity, rate of flow through the reservoir, the geometric features of the reservoir and outlet structures, reservoir operation, as well as on the age of the reservoir; it can also be affected by chemical properties if flocculation occurs. Although trap efficiency of large reservoirs tends to be high, it does not increase linearly – and the useful life of a large reservoir is generally longer than that of a small reservoir, if all other factors remain constant.

In general, the sedimentation process may dictate the lifetime of a reservoir. The lifetime of the reservoir is that time period when the dead space of the reservoir are filled whiles meeting the target demands all the same. It is assumed that, at the beginning of the reservoir operation, the dead space will get filled up first, and then after that, the sedimentation of the serviceable space of the reservoir will follow. Thus, the duration of sedimentation of the dead space is defined by: $t_d = V_d/W$ – where V_d (m^3) is the volume of the dead space and W (m^3/year) is the multiannual average volume of the sediment discharge; and the duration of full sedimentation of the reservoir is $t_f = V_{H,max}/W$ – where $V_{H,max}$ (m^3) is the volume of the reservoir that is associated with the maximum water level. In continental conditions, the typical duration of the full sedimentation is about 20 to 30 years for the local small reservoirs used for drinking and industrial water supply; in the case of larger reservoirs, the expected duration is about 50 years (Pleskov, 1985). If $t_f < 50$ years, then the process of sedimentation could become a serious design parameter; consequently, this factor should be carefully analyzed, because the existence of this type of situation means sedimentation can be very intensive. If t_f is 50 to 200 years, then a far less detailed evaluation will typically be adequate.

The volume of the sedimentation can be evaluated on the basis of measured volume of bed-load and suspended sediment. The distribution of sediment within the reservoir and the retention of sediment depend on sediment size and texture, reservoir inflow and outflow and the size and shape of the reservoir. Estimation procedures for these and related parameters are discussed in the literature by various authors (e.g., Linsley *et al.*, 1982; UNESCO, 1982; Gottschalk *et al.,* 1971; HEC, 1977; Vanoni, 1976; Davis and Sorensen, 1969; Chow 1964).

Reservoir Sediment Storage

The most common procedure for dealing with the sediment problem is to designate a portion of the reservoir capacity as *sediment storage*. This may be viewed as a 'negative' approach that in no way reduces the sediment accumulation problem but merely postpones the date when it becomes serious; nevertheless, this is a reasonable and easy procedure to employ in the storage capacity estimation/design process.

Overall, the fundamental reservoir design question of: 'how large does the reservoir storage need to be, in order to provide for a given demand with an acceptable level of reliability?' calls for need to carefully establish the appropriate relationship between inflow characteristics, reservoir storage capacity, reservoir release, and reliability of operation (Mosaedi, 1998). In this effort, sediment storage and related effects should also be very carefully assessed. In fact, many reservoirs cannot perform as designed because much of their storage volume gets filled with sediment too soon. Bed-load sediment particles may also cause abrasion of turbine blades, tunnels, and gate recess. Furthermore, accumulations of sediments in various parts of the gate structure may affect the operation of these structures. Every effort should therefore be made to minimize these potential shortcomings.

After a site has been selected, the reservoir capacity should be made large enough to create a useful life sufficient to warrant the construction of the dam. Sediment reserve storage is generally estimated separately; this dead storage should be large enough so that it will not be filled before the economic life of the project is over. Indeed, the long-term storage of sediment in a reservoir is stochastic in character and its prediction must – more realistically – therefore be based on the theory of probability. Gottschalk *et al.* (1971) discuss a typical stochastic sediment storage model that is formulated based on Moran's theory (Moran, 1959); the total sediment deposited in the whole period is indicated as an additive process defined on a finite Markov Chain.

It is noteworthy that, the vulnerability of the economic life of a reservoir to sedimentation problems stems from the nonlinear relationship that exists between yield of water and reservoir capacity. Severe reductions in yield of water from a reservoir can result from relatively small changes in reservoir volume – and the reduced storage space could be the result of reservoir siltation.

3.3.6. ADJUSTMENT OF STORAGE ESTIMATES FOR NET EVAPORATION LOSSES

Supplementary evaporation associated with storage reservoirs is the result of the increased water surface following the creation of the reservoir; consequently evaporation from the water body is greater than that from the pre-construction surface. Prior to dam construction, the long-term rainfall-runoff relation for the area that will be flooded by the proposed reservoir can be expressed as follows:

$$R_b = P_b - ET_b \tag{3.10}$$

where R_b = mean annual runoff before reservoir inundation; P_b = mean annual rainfall; and ET_b = mean annual evapotranspiration. After the reservoir is filled, the relation can be expressed as:

$$R_a = P_a - EO_a \tag{3.11}$$

where R_a = mean annual runoff after reservoir is filled; P_a = mean annual rainfall; and EO_a = mean annual evaporation from the water surface. Assuming that the mean annual rainfall before and after construction remains approximately equal, i.e., $P_b = P_a$, then:

$$\Delta E = R_b - R_a = EO_a - ET_b \tag{3.12}$$

where ΔE = mean annual net evaporation loss, and noting that usually $\Delta E \geq 0$.

Anticipated evaporation is a particularly decisive element in design of reservoirs to be constructed in arid regions. Rates of evaporation vary, depending on meteorological factors and the nature of the evaporating surface. The relative effect 'of pertinent meteorological factors is difficult to evaluate at best – recognizing that the rate of evaporation is influenced by solar radiation, air temperature, vapor pressure, wind, and minimally by atmospheric pressure. Since solar radiation is an important factor, evaporation also varies with latitude, season, time of day, and sky condition. Though salinity effects can be neglected in the estimation of reservoir evaporation, any foreign material that tends to seal the water surface or change its vapor pressure or albedo will affect the evaporation. Possible methodologies to employ in evaporation estimations needed in reservoir design include water-budget determinations of reservoir evaporation; energy-budget determinations of reservoir evaporation; aerodynamic determination of reservoir evaporation; combination methods of estimating reservoir evaporation; and estimation of reservoir evaporation from pan evaporation and related meteorological data (Linsley *et al.*, 1982).

Open surface water evaporation can be estimated by using one of the available theoretical and empirical procedures or by applying the annual pan coefficient (p) to tank evaporation data (E_p), as follows (see, e.g., Chow, 1964):

$$EO_a = p \cdot E_p \tag{3.13}$$

Pre-dam evapotranspiration estimates are difficult to determine; one approach is through equation (3.10) above, which yields the following relationship:

$$ET_b = P_b - R_b \tag{3.14}$$

Estimates of mean annual rainfall (P_b) are readily available; mean annual runoff (R_b) can be calculated using data for the catchment, or estimated from regional runoff maps.

The basic requirements for the estimation of evaporation losses based on storage-area relations for a given reservoir are the reservoir storage volume/surface area function – represented by a storage-area relationship (see Figure 3.7 for illustrative sketch); and the average evaporation rate, e_t, for each period. Multiplying the average surface area [m^2] by the loss rate e_t [m] yields the volume [m^3] of evaporation loss in the period. The evaporation loss L_t may then be approximated by (Loucks *et al.*, 1981):

$$L_t = A_a e_t \left(\frac{S_t + S_{t+1}}{2} \right) + A_o e_t \qquad (3.15)$$

where S_t, S_{t+1} = storage volumes [m^3] in the reservoir at the beginning and end of period t respectively; and e_t = average evaporation rate [m] for period t.

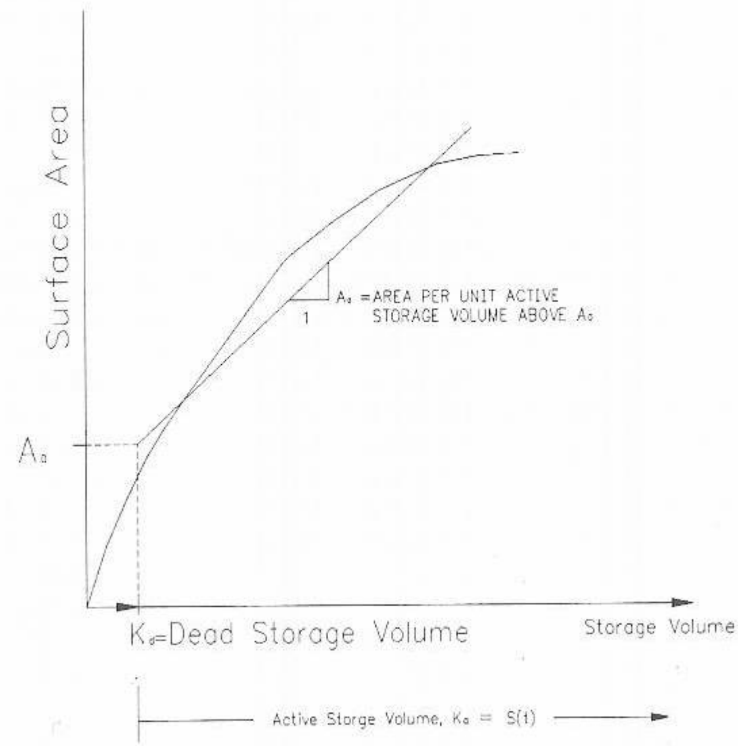

Figure 3.7. A schematic of storage-area relationships and approximation of surface area per unit active storage volume (adapted from Loucks *et al.*, 1981)

In reservoir design, the engineer should be truly concerned with the increased loss over the reservoir site resulting from the construction of the dam, i.e., reservoir evaporation less evapotranspiration under natural conditions. Indeed, it must be recognized that the role of evaporation losses on reservoir design can be rather significant – as illustrated by the shifts in storage requirement curves shown in Figure 3.8. Consequently, the adjustment of storage estimates for net evaporation losses should be carefully evaluated in the reservoir design effort – especially for those projects planned for arid regions. In humid areas, construction of a dam causes only a nominal increase in water loss.

It is noteworthy that evaporation may be reduced by mechanical covers and/or by artificial films if there are no significant wave effects. On the other hand, the presence of certain aquatic plants can bring about tremendous evapotranspiration losses.

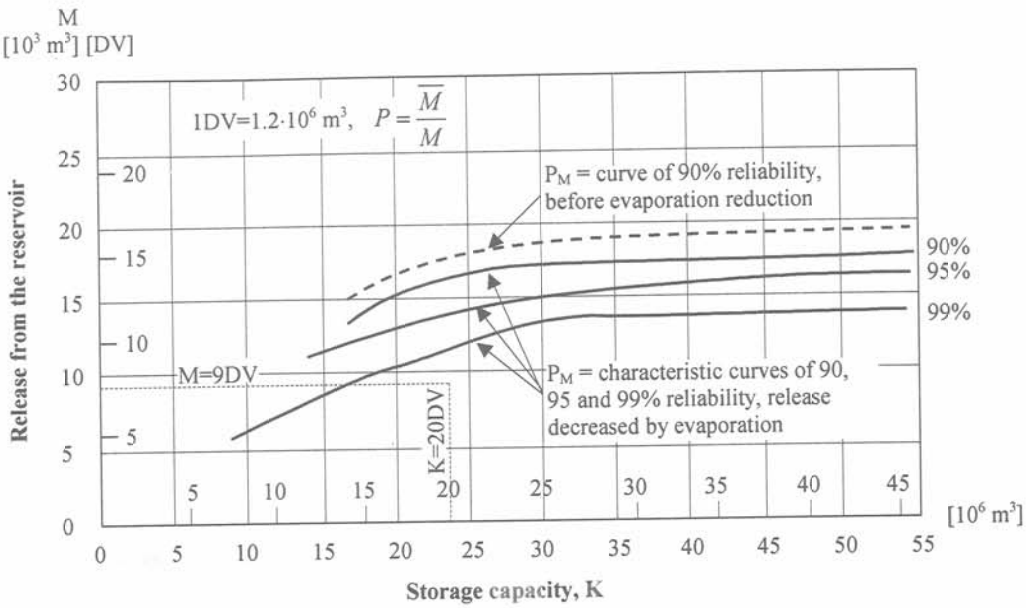

Figure 3.8. Characteristic annual curves with evaporation reductions for the Borjad Reservoir – showing reliabilities of supply

3.3.7. OTHER SECONDARY FACTORS AFFECTING RESERVOIR SIZE-SELECTION

Water losses from a reservoir consist of 'supplementary' evaporation, leakages, and under certain climatic conditions, of water losses due to ice formation. The water losses due to ice formation are connected with the fact that in winter time, when emptying the reservoir, sheets of ice cakes may remain on the shores of the reservoir. Whiles these losses may not be desirable, the benefits derivable from regulation at the reservoir site should offset the hydrologic losses and the costs of reservoir construction and operation – otherwise, the reservoir should not be built at all.

Analysis of Seepage Problems

Further to evaporation losses, the use of reservoirs to temporarily store streamflows often results in a net loss of total streamflow due to seepage. Leakage losses from reservoirs are composed of seepage through the valley bottom and slopes, and through the layers below the hydraulic structures. The loss due to seepage beneath the dam structure is a function of the water head, *h*, and seepage entry surface area, A_s (which in turn is dependent on the storage volume, S). Various more exact and complicated hydraulic models may be used for the calculation of seepage losses; however, it seems that a simplified model based on the Darcy law can be used for a stochastic description of seepage. Making the simplifying assumption that the seepage below the dam follows Darcy's law, then:

$$q = k \bullet i \bullet A_s \tag{3.16}$$

where q = seepage discharge (m^3s^{-1}); k = Darcy's permeability coefficient (ms^{-1}); A_s = surface area at entrance of seepage water (m^2); and $i = \Delta h/L$ = hydraulic gradient (dimensionless), where Δh is the change in reservoir level (m) over distance L (m) along the dam axis. The seepage loss, q, under the dam can be estimated for every reservoir level prevailing at one time or the other. Reservoirs may also be constructed with the drain systems that collect seepage water to the tailrace for volumetric measurements; thence, estimates of the average values of A_s and k can be obtained, and a correction factor found for its previous estimates. Modern systems would incorporate automatic monitoring mechanisms.

In practice, seepage through the layers of the valley can be determined by special hydrogeological explorations. In the absence of such investigation results, data from reservoir sites that have comparable hydrogeological conditions can be used as basis for the seepage assessment. During preliminary design calculations, some specified percentage of the available reservoir volume may be used as the probable seepage for the following different geological situations:

a) 'Good Geological Conditions' – comprising clay loam bed of great strength and close proximity of groundwater on the slopes, use 5 to 10 per cent.
b) 'Average Geological Conditions' – i.e., soil of fair to marked perviousness (in conjunction with the use of anti-seepage measures), use 10 to 20 per cent.
c) 'Difficult/Poor Geological Conditions' – consisting of pervious ground, use 20 to 40 per cent.

In general, where groundwater levels at the dam site are higher than the maximum water levels in the reservoir, no seepage losses from the reservoir would be anticipated. On the other hand, when groundwater levels of the selected dam site are lower than the maximum value of the water level in the reservoir (especially in the case of very coarse sand and coarse gravel or cobbles), seepage losses can be very high – in which case the feasibility of the whole project becomes questionable.

Freeboard Requirements for Wind and Wave Action

Freeboard is the vertical distance between a referenced water surface and the top of the dam structure; it is used to insure the safety of structures against overtopping in the event of the occurrence of the design flood. A dam must normally be constructed with a freeboard above the maximum water depth, H_{max}, associated with the design capacity, K; this would then allow the reservoir/dam to safely pass the floods that would be routed through it, as well as prevent any wind-generated waves from jeopardizing the dam crest.

The purpose of freeboard is to act as a safety measure against overtopping of a structure due to an adverse coincidence of physical forces acting to force the water level to rise above the maximum still water level. Factors that can act independently or in concert to cause this relative rise in water surface above the normal still water design level include sustained high wind, long fetch, ice in spillways, and floods exceeding the spillway design flood.

A typical approach to use in this evaluation of the freeboard requirements would be to determine the crest elevation on the basis of either the highest flood discharge passing the reservoir, or the strongest winds – since the probability of coincidence of these two independent random events is rather small (and can be neglected for all practical purposes). A discussion of some relevant mathematical relationships for estimating the wave height is elaborated elsewhere in the literature (see, e.g., V. Nagy, 1990; Davidan, 1989; Ivanov, 1989; UNESCO, 1982, 1974; USACERC, 1977).

The highest flood flows usually pass reservoirs within a few hours. In Europe, for instance, rarely does the duration of the strongest wind exceeds 4 to 6 hours. The coincidence of the extreme values of these two random variables, flood and wind, can be neglected for all practical purpose – the probability of coincidence of these two independent events being of a small order of magnitude (about 10^{-4}). Thus, a reasonable probabilistic approach to determining the crest elevation is based on one of these two variables, but checked against the other. The distribution functions of reservoir stage and the wave height must be determined in order to establish the crest elevation.

In design practice, the value of the dam crest elevation is usually calculated as follows:

$$H_{\max} = H_{av} + \Delta h + h_w + a \tag{3.17}$$

where H_{av} is the average water depth, m; Δh is the wind-caused water level rise, m; h_w is the height of the wave, m; and a is the safety factor (a ~0,5 m). The maximum value of h_w is usually selected at the probability of occurrence level of between 0.01 and 0.05, depending on the importance of the structure. For the purpose of a preliminary evaluations of the wave height, where the wind speed, v~20 m/s, Figure 3.9 can be used. A more detailed analysis and empirical results are presented elsewhere in the literature (e.g., USACERC, 1977; Davidan, 1989). For shallow reservoirs where the length of the reservoir, L < 100 km, the wind-caused 'super-elevation' of the water surface can be neglected by simply adding a safety value of depth (a), depending on the importance of the reservoir. This value is usually $a \cong 0.5$ m. Subsequently, the dam crest elevation is determined by the height of the wind-generated waves.

Ultimately, the amount of freeboard is partially based on experience – and should be higher when damage potential is greater. Overall, overtopping of the non-overflow sections of the structure should be prevented if such overtopping is likely to cause failure of the embankment.

3.4. Hydrologic Data Requirements and Analyses