Chapter 11

RESERVOIR OPERATION

The objectives of this chapter are:

- to explain the problem and issues in reservoir operation,
- to explain conventional and rule curve based approaches of reservoir operation,
- to explain the procedures for operation of a system of reservoirs for multiple purposes,
- to illustrate the systems analysis technique with a real-life example.

After structural facilities, such as dams, barrages, hydropower plants, etc., come into being, the benefits that could be reaped depend to a large extent on how these facilities are operated and managed. The efficient use of water resources requires not only judicious design but also proper management after construction. By far, most good dam sites have already been developed in many countries. Due to various reasons, there is slowdown in construction of new projects and the construction cost of new projects is increasing with time. Biswas (1991) estimated that the unit cost of water from the next generation of municipal water supply projects would usually be 2 to 3 times higher than from the present generation. It is, therefore, imperative that all projects are managed in the best possible manner. A conceptual depiction of the need of regulation to meet the requirements of the society is given in Fig. 11.1.

Guidelines for operation of a reservoir have to be developed in the planning stage of a project. Later on, these are refined on the basis of actual operational experience. Such schedules range from rigid rules which are to be exactly followed to flexible guidelines that permit considerable leverage to the operator. Such schedules can be in graphical, tabular, or narrative form or a combination of these.

Reservoir operation is an important component of water resources planning and management. After construction, detailed guidelines are given to the operator to enable him to take appropriate decisions. A reservoir operation policy specifies the amount of water to be released from storage at any time depending on the state of the reservoir, level of demands and any information about the likely inflow to the reservoir. The operation problem for a single-purpose reservoir is to decide the releases to be made from the reservoir so that the benefits for that purpose are maximized. For a multipurpose reservoir, additionally, it is also required to optimally allocate the release among purposes. The complexity of the problem of reservoir operation depends on the extent to which the various intended purposes are compatible. If the purposes are compatible, less effort is needed for coordination. At this stage, it is helpful to discuss the conflicts among various purposes.

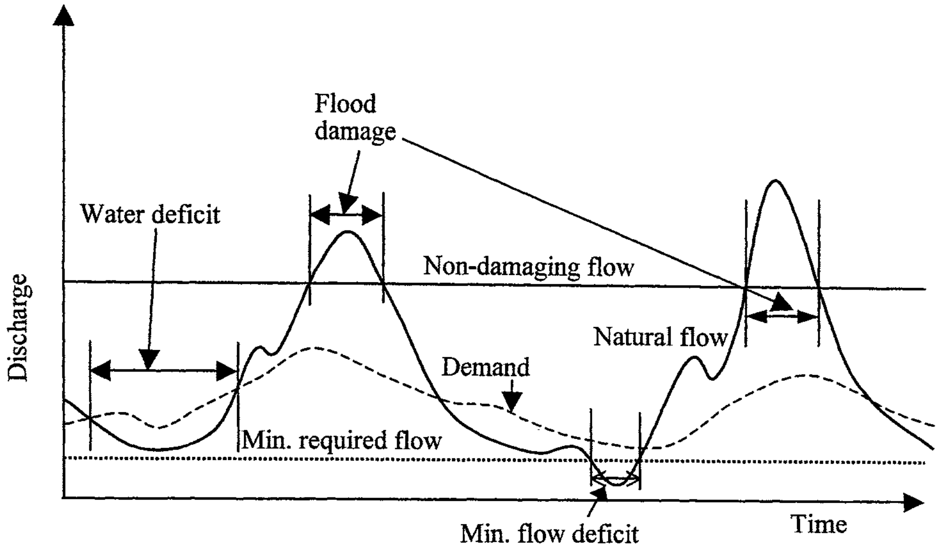

Fig. 11.1 The need of regulation to meet the requirements of the society.

11.1 CONFLICTS IN RESERVOIR OPERATION

While operating a reservoir that serves more than one purpose, a number of conflicts arise among demands for various purposes. The conflicts that arise while operating a multipurpose reservoir may be classified as follows:

a) Conflicts in Reservoir Space

These conflicts occur when a reservoir (of limited storage) is required to satisfy divergent purposes, for example, water conservation and flood control. If the geological and topographic features of the dam site and the funds available for the project permit, the dam of sufficient height can be built and storage space can be clearly allocated for each purpose. However, this seldom being the case, multipurpose reservoirs with shared space are developed. The satisfaction of conservation purposes requires the reservoir to be filled to the maximum extent possible, whereas the objective of flood control is best met when sufficient vacant space is available in the reservoir. Therefore, while regulating such a reservoir the crucial decision is whether to fill the reservoir or keep it vacant. A full reservoir allows reaping higher benefits by satisfying conservation purposes but at the same time, there is a higher risk of flood damages. On the other hand, an empty reservoir can moderate floods more effectively but if the flows are not up to the expected volume, the reservoir may remain vacant and consequently the conservation purposes will suffer.

b) Conflicts among Purposes

Within the conservation purposes also, conflicts can arise when the pattern of water use is different and the requirement of storage space for one purpose is not in conformity with the other purpose. For example, irrigation demands may show one pattern of variation, depending on the crops, season and rainfall, while the hydroelectric power demands may have a different variation. The water required for consumptive uses like irrigation, municipal water supply cannt be shared with any other use. The conflicts in daily discharge are also experienced in a reservoir which serves more than one purpose. If a reservoir is for a consumptive use and hydroelectric power generation, the releases for the two purposes may vary considerably in the span of a day. Similarly, the benefits from the use of a reservoir for recreation are high during summer but the irrigation demands may also be the highest during this period requiring drawdown of the reservoir level.

c) Conflicts within the same Purpose

A deficit of water can be distributed over time in different ways. A typical decision is whether the supply should be cut now so that there is a small deficit for a longer period or postpone the cut for the future and risk a bigger shortage albeit for a shorter time. The impact of these two decisions will be different in different situations and will also depend on the use of water, viz., irrigation, municipal water supply, and so on.

In a multi-reservoir system, the decision-maker also has to allocate releases as well as deficits among reservoirs. Evidently, none of the common uses of water are 100% compatible with each other. The operation policy should guide the operator in satisfactorily resolving these conflicts. It may be emphasized here that the key word in regulation of a multi-purpose reservoir is “compromise”.

11.2 CRITICAL ISSUES IN RESERVOIR OPERATION

According to James and Lee (1971), the following six issues need to be optimally resolved while developing an operation policy of a reservoir:

1. *Use of Flood Storage*: Whether flood inflows should be stored to reduce current damages or released to provide additional storage space in case new rains produce even greater flows.
2. *Use of Total Storage*: Whether storage space should be filled to save water for beneficial use or emptied to contain potential floods.
3. *Release of Stored Water*: Whether water stored within the reservoir should be released

for present use or retained for use during possible future droughts.
4. *Release by Reservoir*: How much of the water to be released for beneficial use should come from each reservoir in which water is stored?
5. *Use of Available Water*: How the water released from the reservoir should be divided among various potential uses.
6. *Release Elevation*: Whether the released water should be taken from near the surface or from some elevation deeper within the reservoir.

The above issues are briefly discussed in what follows.

11.2.1 Use of Flood Storage

The two main objectives of flood management are: (i) to minimize the downstream damages, and (ii) to ensure dam safety. An important decision while regulating a flood control reservoir is whether flood flows should be stored in the reservoir to control the current flood or be let out to provide additional storage space in case a bigger flood occurs. This question arises when the storage level in the reservoir is in the flood control zone. The controlling parameters are the available flood storage, the current and forecasted inflows, the safe carrying capacity of the downstream channel, and the status of other reservoirs in the system. While making releases, the current flow at the damage center and the likely contribution from the catchment downstream of the reservoir up to the damage center should also be considered. It might be prudent to release at a rate equal to the safe capacity of the downstream channel less local flows, if the reservoir is in the flood storage zone. If the forecast indicates the possibility of larger floods, releases slightly exceeding safe carrying capacity of the downstream channel can be made to avoid severe damages subsequently. There is an economic trade-off between the increase in the downstream damages caused by larger releases and the increase in the expected value of future damages caused by less storage space available to contain subsequent flows. To properly manage floods, the operator should know the damages which high reservoir releases can cause in the downstream area.

Reliable precipitation and inflow forecasts are not always available to the dam operators. Therefore, the flood control regulation schedule for a reservoir is normally developed based on the information that is likely to be available with the operator at the dam site, viz., the current inflow rate, reservoir elevation and rate of rise/fall, and volume of inflow that can be expected in a flood.

11.2.2 Use of Total Storage

The second important issue is whether the storage space should be filled by storing water for some future beneficial use or be emptied to absorb likely floods. This question primarily deals with the operation of a reservoir in wet season when the objective may be to moderate the potentially dangerous floods whenever there is a significant probability of their occurrence and to store water to the capacity of the reservoir for beneficial use in the dry season. The controlling parameters are the amount of water currently in storage, the vacant space for flood control, the value of stored water, and the risk of flood.

The usual practice is to keep the reservoir empty in the first few weeks of the flood season and gradually fill it up as the season progresses. The rule curves for a multipurpose reservoir are designed so that the release from the dam should not be very and high reservoir is full at the end of the filling season. However, if the flows in the wet season are not as expected, the reservoir may not be completely filled. Clearly, a better strategy would be to fill the reservoir sufficiently at the first available opportunity. If a larger flood is expected subsequently, the storage could be depleted to create the requisite vacant space and the storage is repleted again when the flood begins to recede. This procedure can be effectively implemented if reliable forecasts of rainfall and inflows are available.

The economic trade-off is between the value of the additional water stored within the reservoir and the additional flood damages if vacant space is not available when the flood occurs. Water can be stored in the joint-use space of a reservoir for other purposes as long as the flood control operation is not hindered. The water stored for conservation purposes is released based on demands. Thus, the main operating decisions that require the operator's attention are the amount of flood control space that ought to be available at any time and size of the releases required to create additional space, if necessary.

11.2.3 Release of Stored Water

The third question is important for operation during a dry season to ensure that the water would be available to meet the demands during the most critical year. There is a trade-off between the benefit received from additional water when put to present use and its expected value in future. The trade-off is in terms of the probability of wastage of water if the reservoir spills versus the probability of a severe drought which would make the stored water very valuable.

11.2.4 Release by Reservoir

The fourth question pertains to the apportionment of release among different reservoirs in a system. How the water to be released for beneficial purposes should be divided among reservoirs ? For flood control purposes, this will depend on the vacant storage space available in the reservoirs, conditions in the downstream reaches of each of the reservoirs as well as the likely inflow to each of the reservoirs. If the reservoirs serve conservation purposes, intuitively more water should be drawn from the reservoir that is likely to receive more inflow. For the best results from the operation of a reservoir system, it is necessary that the operation policies are jointly developed.

11.2.5 Use of Available Water

This issue pertains to the allocation of water among different uses. James and Lee (1971) suggest a division of water among the various uses until its marginal value for each is equal.

11.2.6 Release Elevation

Primarily due to temperature variations, reservoirs tend to stratify into zones of different density. The warmer upper layer of a lake is known as epilimnion (it might be colder during winters). The temperature changes rapidly with depth in the intermediate metalimnion zone. The lowest zone where water is usually densest and coldest has the lowest dissolved oxygen concentration and the largest concentration of sediments is known as the hypolimnion zone. These three zones are well defined only during warmer summer months; the reservoir is nearly isothermal during other seasons.

The important water quality parameters from a reservoir operation point of view are temperature, biochemical oxygen demand, dissolved oxygen, and suspended sediments. If the quality of released water is an important consideration, it is necessary to have outlets that can draw water from different elevations. The operator can then let out water from the appropriate outlet(s) to ensure that the water of the desired quality is supplied.

With the above background, the various approaches to develop policies for operation of reservoirs will be discussed in the following.

11.3 BASIC CONCEPTS OF RESERVOIR OPERATION

The drawdown refill cycle of a reservoir is usually 12 months long except when the reservoir capacity is large in relation to streamflows. The cycle may extend over many years in arid regions. In many regions of the world, the refill periods (when inflows are more than the demands and therefore extra water is stored in the reservoir for later use) and drawdown periods (when inflows are smaller than the demands and therefore water is withdrawn from storage to meet various demands) are distinctly separated. For example, in monsoon climate, high flows occur during certain calendar months only. Many reservoirs receive a significant portion of their annual flows through snowmelt. In addition, benefits from operation of reservoirs considerably improve when reliable weather and inflow forecasts are available.

A reservoir is operated according to a set of rules or guidelines to store and release water depending on the purposes it is required to serve. The decisions regarding releases in different time periods are made in accordance with the available water, inflows, demands, time of the year, etc. Many operation rules are based on intuition and common sense. For example, in a multi-reservoir operation, the consumptive demand may be met from the reservoir that is nearest to the demand point so as to minimize transit losses and wastage. Likewise, in irrigation operation, the manager may release water to save the standing crop from serious damage and take the risk of shortage of water for a future crop.

For reservoirs which are designed for multi-annual storage, the operation policy is based on long term targets. The estimates of water availability are made using long-term data. The requirements for conservation uses are worked out by projecting the demand data. The magnitudes of releases for the uses which are to be served from storage on a long-term basis are determined and the reservoir is operated accordingly. In periods of droughts, based on pre-specified priorities, the supply for some uses is curtailed keeping in view the minimum demands of each purpose. Consideration is given to the maintenance of essential services even if it is at the cost of agriculture and industrial production.

Many basins in cold countries experience floods when the snow melts. It is possible to fairly accurately predict the runoff volume during this period by using snow surveys data and storage may be allocated to ensure desired flood protection. One can be quite sure that this space will be filled by the time flood season is over. However, such a long range forecasting with desired reliability is not possible for rain-fed rivers in monsoon climate, and a calculated risk is taken while allocating storage space for flood control.

11.3.1 Long-range Planning Schedules

The long-range schedules are developed during the planning stages typically to estimate the project reliability, likely benefits or the type and extent of demands that can be met. Such schedules are also developed during the system expansion, i.e., when a new reservoir is to be added to an existing configuration. For conservation purposes, long-term (monthly or annual) data are commonly used for this type of analysis. Suitable assumptions are made to keep the problem tractable. For example, normal values of the amount of evaporation may be used to estimate evaporation losses from the reservoir. Despite their crudeness, the results of such studies can provide useful insights into the problem and form the basis of refined and detailed studies in the latter stages of planning and eventual operation.

11.3.2 Rigid Operation Schedules

Rigid operation schedules are needed for the eventuality when decisions are to be taken urgently but the required detailed data are not available or there is not enough time to analyze them. Such a situation is most likely to arise in flood management. An example of extreme rigidity in operating schedule is a single purpose flood control reservoir with ungated spillway(s). In a way, the operation policy is built into the structure through the elevation of the spillway crest and its shape. Rigid schedules for flood control operation of a gated reservoir are needed for use by non-technical staff at the dam site. Such schedules are formed on the basis of the study of the design flood or the probable maximum flood. Therefore, these schedules are based on data, such as the reservoir elevation, river stage at a downstream point, reservoir inflow, and the rate of change of reservoir elevation which are available to the operator even when all means of communication break down.

As too much rigidity is a hindrance in realizing the maximum benefits, some flexibility in operation is preferred. While there are no two opinions that the operator should be provided with a detailed set of instructions covering all the situations that are likely to arise, he should have enough flexibility to fine tune the decisions based on specific at-site conditions. Usually, the day-to-day reservoir operation is based on current and forecast of stream flow, status of demands, and precipitation outlook. A crucial factor in effective operation, particularly in case of floods, is the availability of reliable forecasts. Many day-to-day decisions are taken based on judgment and supported by knowledge gained by the study of past events.

11.3.3 Standard Linear Operating Policy

The simplest of the reservoir operation policies is the standard linear operating policy

(SLOP), graphically represented in Fig. 11.2. According to this policy, if in a particular period, the amount of water available in storage is less than the target demand, all the available water is released. If the available water is more than the target demand but less than target demand plus available storage capacity, the release equal to the target demand is made and the excess water is stored in the reservoir. In case, even after making releases equal to the target demands, there is no space to store the excess water, all the water in excess of the maximum storage capacity is released.

Let A_w represent the available water and T the target demand. Mathematically, the SLOP can be expressed as:

$$\begin{array}{lll} \text{If} & A_w \le T, & \text{Release} = A_w \\ \text{If} & T < A_w \le S_{max} + T, & \text{Release} = T \\ \text{If} & A_w > S_{max} + T, & \text{Release} = A_w - S_{max} \end{array} \qquad (11.1)$$

The reservoir will be empty in the first case and full in third. The zone of feasible releases lies between the lines of full and empty reservoir. The SLOP is a one-time operation policy without relation to the release of water at any other time. This type of time isolated releases of water is neither beneficial nor desirable. The water beyond the target demand in any period has no economic value. Although this policy is frequently used in planning studies, it is not used in day-to-day operation due to its rigidity. Some of its drawbacks can be minimized by introducing rationing in the event of a deficit. For instance, if the available water is less than the demands for the current and next three months, the release may be reduced by some amount, say 25%. Depending upon the circumstances, rationing may be introduced in several stages; the number of stages and the extent of rationing can be decided by simulation. The double line in Fig. 11.2 shows one such possibility. The resulting rule was termed *rationing rule* by ReVelle (1999).

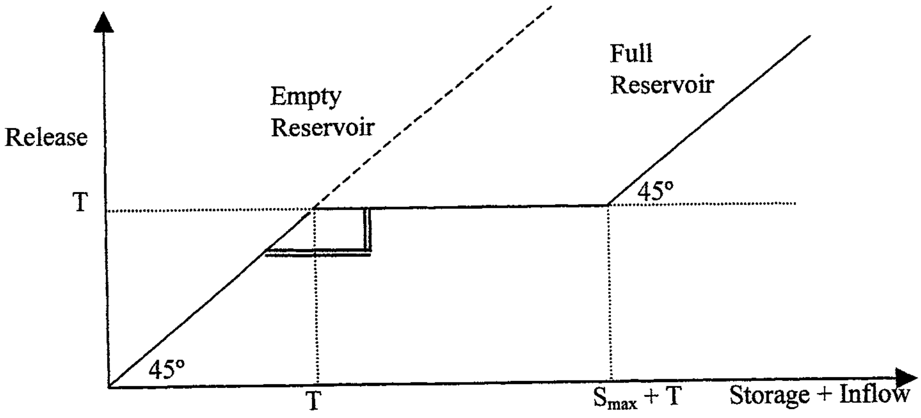

Fig. 11.2 Graphical representation of the Standard Linear Operation Policy.

11.4 RULE CURVES

A rule curve or rule level specifies the desired storage to be maintained in a reservoir as closely as possible during different times of the year while trying to meet various demands. The rule curves are generally derived by operation studies using historic or generated flows. Here the implicit assumption is that a reservoir can best satisfy its purposes if the storage levels specified by the rule curve are maintained in the reservoir at different times. The rule curve as such does not give the amount of water to be released from the reservoir. This amount will depend on the inflows to the reservoir and the demands for various purposes. Different rule curves may be developed for different purposes such as municipal water supply, irrigation, hydropower generation and for flood control.

11.4.1 Derivation of Rule Curves

The derivation of rule curves depends on the type of the reservoir and the purposes to be served. A reservoir may be classified either as a seasonal reservoir or a multi-annual reservoir. The storage of a seasonal reservoir is utilized to carry water from the wet season to the dry season, whereas multi-annual reservoir storage is used to carry water from a wet period to a subsequent dry period which could occur several years later. Consider the case of a reservoir with seasonal storage serving conservation needs. If this reservoir is able to meet the demands during the critical year, it will be able to do so in all other years.

The streamflow of a river during the driest year on record and the water requirements have been plotted in Fig. 11.3(a). Assume that the reservoir is full at time A. From A to B, the demands exceed the natural inflow and hence the reservoir will deplete and will be empty at B. From time at A onwards, the inflow and demand curves diverge and the cumulative difference is maximum at B. This difference represents the required storage capacity. The mass curves of inflows and demands have been plotted in Fig. 11.3(b). In this figure, at point B, the reservoir is empty. From this point, the demand mass curve is plotted backwards in time and curve BE is obtained which is nothing but the mass curve AB of demands extended to the left. The vertical ordinates between the inflow mass curve EAB and demand mass curve EDB represent the volume of water which is in storage during the period from E to B. These vertical ordinates have been plotted against time in Fig. 11.3(c) and the resulting curve is the rule curve. This rule curve represents, in a reverse order of time, the accumulation of the deficiency between demands and available streamflow during the critical period. Since this analysis has been performed for the driest year on record, it can be safely concluded that whenever there is more water in the reservoir than specified by the rule curve, there is no danger of failure of the reservoir.

Since a rule curve depends on the flow pattern in a critical year, it would be desirable that rule curves for other near-critical years are also prepared. When these curves are plotted on the same graph, these will cross one another at several places. Finally, a smooth enveloping curve is drawn and is the requisite rule curve. The rule curves for carry-over storage can also be prepared in a similar way. The above is the graphical technique for preparing rule curves. A computer-based approach for preparation of rule curves is explained in a later section.

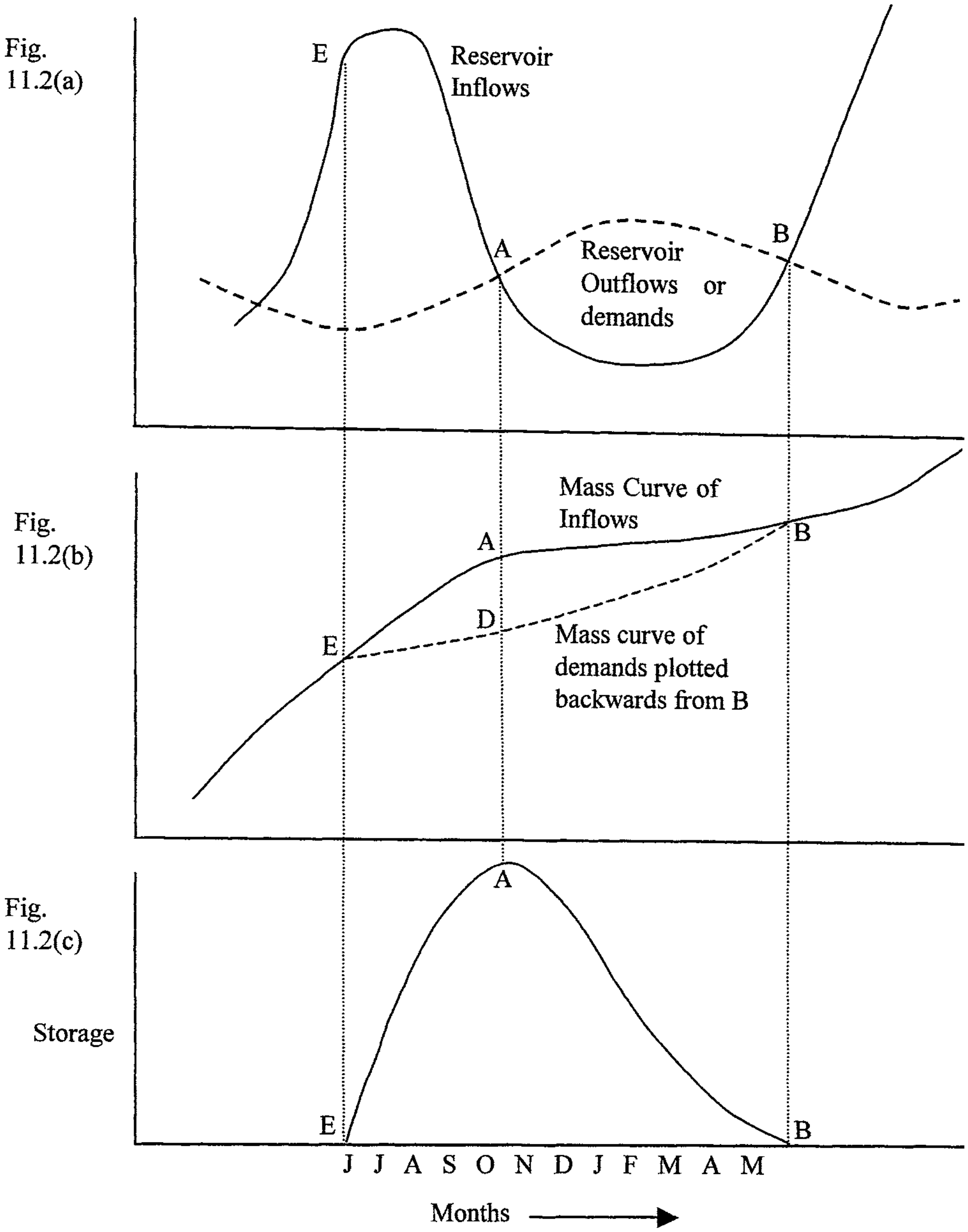

Fig. 11.3 Development of rule curve for conservation operation [adapted from Kuiper (1965)].

11.4.2 Operation of a Reservoir Using Rule Curves

While operating a reservoir with the help of rule curves, there are several possibilities. If the water level at any time is above the elevation stipulated by the rule curve at that time (i.e., enough water), releases are made to meet all conservation demands. If the available storage is in the vicinity of that indicated by the rule curve, the release of water should be restricted such that the storage does not fall appreciably below the rule curve level. If for some reason, the level in the reservoir is much below the rule curve, the release should be curtailed with attempts to return to the rule curve level at the earliest.

The rule curves implicitly reflect the established trade-off among various project objectives in the long run. For short-term operations they serve as a guide. Thus, the reservoir operator has flexibility to decide the releases so that the long-term objectives are fulfilled to the maximum possible extent. A reservoir operation schedule which gives some leverage to the operator to use his judgment, and experience is termed as flexible schedule. Note that the rule curves only specify the ideal levels to be maintained and the operators can use their experience and judgment to distribute excess or deficit over space and time to maximize benefits. The release decision may also incorporate the relative priority among various uses and in case of deficits, the higher priority demands are met first.

In order to provide further flexibility in operation, different rule curves may be specified for different circumstances. For example, there can be three different rule curves – one for a normal year in which reservoir inflows are close to the average flow (within ±20% of the average), another for a dry year (inflow below 80% of the average), and the third for a wet year (inflow greater than 120% of the average). In situations where it is possible to forecast floods or snow melts, conditional rule curves can be defined. These rule curves may be presented either in the form of tables or graphs and show the desired reservoir levels as a function of the expected inflows. Conditional rule curves may be defined for the entire water year or a part thereof.

Many times due to various reasons, such as low inflows, minimum requirements for demands etc., it is not possible to adhere to the rule curve. In case of deviations, there are several ways to return to the rule levels. One way is to return to the rule curve by curtailing the release beyond the minimum required if the deviation is downward or making releases at higher rates if the deviation is upwards.

The rule curves are developed using past streamflow and demand data which will not be repeated in future. Therefore, there is a possibility of improvement over rule curve-based operation. Of course, this requires detailed input data, better models, infrastructure, and trained personnel. But the significant amount of larger benefits which can be reaped by improved regulation makes their use attractive for major reservoirs, particularly when the resource are limited and have to be allocated among a number of competing users. Several such approaches are discussed later in the chapter.

11.4.3 Concept of Storage Zoning

As discussed in Section 10.9.2, the entire reservoir storage space can be conceptually divided in a number of zones by drawing imaginary horizontal planes at various elevations. The sizes of these zones need not be constant and can vary with time as shown in Fig. 11.4.

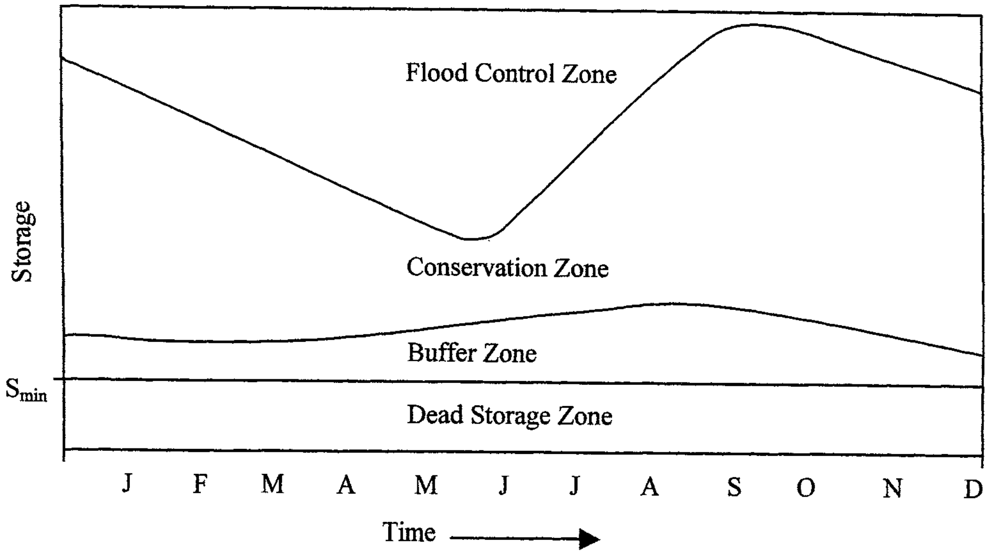

Fig. 11.4 Variation of Reservoir Zones with Time.

During the actual operation, the reservoir managers are expected to maintain the reservoir level in the specified zones. This conceptual division of a reservoir into a number of zones and the rules governing the maintenance of storage levels in a specified range are based on the assumption that at a specified time, an ideal storage zone exists for the reservoir and benefits can be maximized by keeping the storage in this zone. This concept is in some way akin to a rule curve with the added advantage that this approach gives more flexibility to the decision-maker who can carefully steer the storage level within the specified zone to maximize gains. In addition, the rule governing the maintenance of the reservoir level in a particular zone may be conditioned upon the hydrologic state of the system. Thus, the reservoir operator may be advised to keep the level in one zone if streamflow is X and in another zone if the flow is Y.

The common operation policy is to release as much water as possible irrespective of the damages in the downstream area when the reservoir is in the spill zone, to release at the maximum non-damaging rates when the reservoir is in flood control zone, and to bring the reservoir to the top of the conservation zone at the earliest possible time. The release from the conservation zone depends on the requirements of water for various purposes intended to be met by the stored water and the day-to-day releases may be adjusted based on the anticipated inflow and the future requirements up to the end of the operating horizons. When the available water is expected to be less compared to the demand, releases may be curtailed. Broadly, a rule curve also aims at this type of operation.

In the context of multiple reservoirs, zoning offers some flexibility in operation of individual reservoirs. Sometimes, defining sub-zones within the conservation zone provides further flexibility. While managing multiple reservoirs, attempt is made to balance the storage level in different reservoirs, i.e., at any time all the reservoirs are maintained in the same zone to the extent possible. This type of operation is necessary to restore balance among reservoirs after an unexpected or extreme hydrologic event.

There are three approaches for such balancing of reservoir contents. The first, known as the "equal function" policy, is to maintain keeping all reservoirs at their same zonal position, i.e., at a level where the percentage filling of the zone is equal for all the reservoirs. The second one is based on a reservoir ranking or priority concept. Each reservoir is assigned a priority. The entire water in a zone of the lowest priority reservoir is utilized before drawing water from the next lowest priority reservoir, and so on. The third concept is based on a "storage lag" policy. Withdrawals from some reservoirs are begun before drawing water from the same zones of other reservoirs. After a certain volume has been released from the initial group of reservoirs, releases are made from all reservoirs, maintaining the percentage difference of available zone volume. This policy is followed to provide a readily available reserve of water if corrections in inter-reservoir balancing are needed after an unexpected or extreme hydrologic event.

Conditional Rule Curves

Conditional rules have also been used to regulate multiple-reservoir systems. These policies define reservoir releases not only as a function of the existing storage volumes and the time of the year, but also as a function of the expected natural inflows into the reservoirs for some pre-specified time period in the future. Such policies can be described as functions, in tabular form, or as a diagram. For the reservoirs that receive substantial snowmelt, the winter snow depth is a typical input.

The multiple zones and sub-zones and operating rules are prescriptive in character as compared to the simple curve. Defining flow ranges further guides in operation. These ranges for the individual channels downstream of the reservoirs can be defined as a function of the upstream storage volume. Loucks and Sigvaldason (1980) defined three ranges as shown in Fig. 11.5. The *Normal flow range* is considered ideal and the flow should be in this range as long as all the upstream reservoirs are within their respective ideal zones. The *Extended range* is the enlarged range of flows that could be utilized if one or more upstream storage volumes are either in flood control or buffer zone. The *Extreme range* is the further enlarged range of flows that could occur if one or more of the upstream storages are in either the spill or inactive zone. The size and extent of these ranges can be a function of time. With multiple zoning for storage volumes and flow ranging for channel flows, there is less need for operator judgment when balancing reservoir levels with channel flows and keeping the system within the restrictions imposed by these zones or levels and flow ranges.

Sometimes, the water level of a reservoir is fluctuated over a small range to reduce the incidence of diseases. Such fluctuations destroy breeding habitat of mosquitoes.

11.5 OPERATION OF A MULTI-RESERVOIR SYSTEM

The discussion so far was limited to operation procedures for a single reservoir. It is well known that the benefits from the joint operation of a system of reservoirs can be

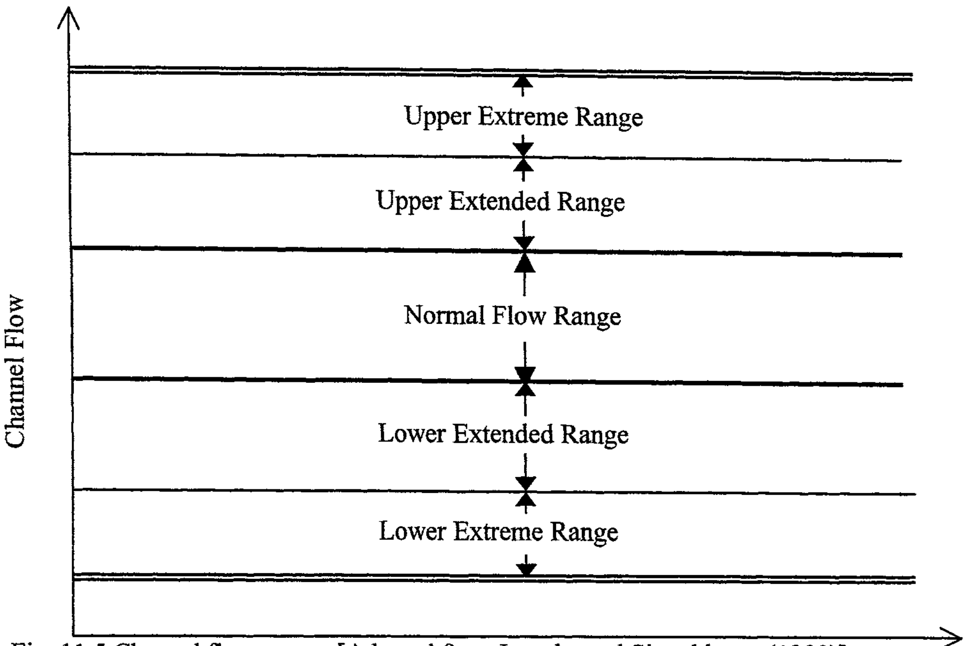

Fig. 11.5 Channel flow ranges [Adapted from Loucks and Sigvaldason (1980)].

substantially larger than the sum of benefits obtained from the operation of individual reservoirs. A system may consist of reservoirs in series, in parallel, or a combination. Approaches to develop operation policies for a system of reservoirs are discussed in the following. Some of these operational policies are developed by intuition and are anticipatory. But this does not diminish their utility and effectiveness.

11.5.1 Reservoirs in Series

Consider a system of two reservoirs in series as shown in Fig. 11.6. A complex system can be decomposed into this simple configuration. The diversion demand D_1 can be met only by reservoir 1 while demands D_2 and D_3 can be satisfied by both reservoirs. The rules for refill and drawdown of reservoirs in series for various purposes are given in Table 11.1.

The reservoirs shown in Fig. 11.6 can serve conservation demands best by minimizing the uncontrolled outflow of water from the system. The spill from any reservoir, except the lowest, can be captured by a downstream reservoir. Thus, the most upstream reservoir should be filled up first (subject to the availability of inflows), followed by the reservoir just downstream to it, and so on. This strategy permits capture of spills from the upstream reservoirs in the system itself.

During the drawdown season, where the natural streamflows are small in comparison with the demands, the most downstream reservoir should be drawn down first and so on. The demands at a location are met by the immediately upstream reservoir before using any other upstream reservoir. This rule can be bypassed if due to various reasons, such as topography, system configuration, it is not possible to meet all the demands by all reservoirs. The relative magnitudes of water loss due to evaporation and seepage from various reservoirs should be considered while applying these rules.

Table 11.1 General rules for operation of reservoirs in series [adapted from Lund and Guzman, 1998].

Purpose	Refill period	Drawdown period
Water supply	Fill upstream reservoirs first	Withdraw from downstream reservoirs first
Flood control	Fill upstream reservoirs first	Withdraw from downstream reservoirs first
Energy storage	Fill upstream reservoirs first	Withdraw from downstream reservoirs first
Hydropower production	Maximize storage in reservoirs with greatest energy production per unit of water	Maximize storage in reservoirs with greatest energy production per unit of water
Recreation		Equalize marginal recreation improvement of additional storage among reservoirs

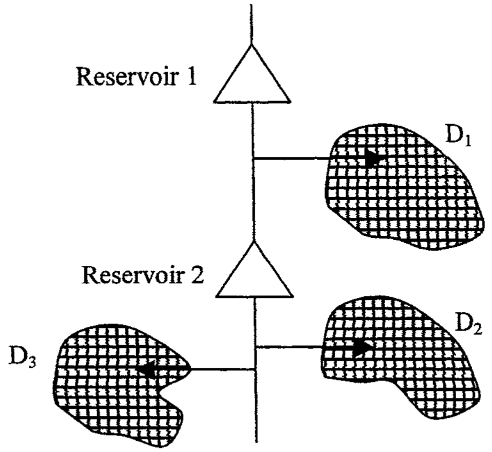

Fig. 11.6 A system of two reservoirs in series.

If these rules are applied to the system shown in Fig. 11.6, demand D_1 is met from reservoir 1 (meeting this demand from reservoir 2 may involve pumping) and demands D_2 and D_3 are met from reservoir 2. When there is not enough water in reservoir 2, water from reservoir 1 is released (assuming that it has enough water) to meet demands D_2 and D_3. This rule ensures minimization of spills from reservoir 2. The spills of reservoir 1 can be captured by reservoir 2.

11.5.2 Hydropower Reservoirs

Water stored in a hydropower reservoir provides assured supply as well as hydraulic head. During the filling season, the aim usually is to have as much energy (in form of water) as possible stored in the system at the end of the season. Since the water stored in an upstream reservoir (higher elevation) has higher potential energy, the upstream reservoirs in a series should be filled first. After generating at an upstream reservoir, the water can be captured in a downstream reservoir where it again generates energy. The same logic also holds good for any spill from an upstream reservoir. Of course while storing water, one has to also examine the compatibility of uses of water. The storage of water for energy generation and other conservation uses may be compatible but will have conflict with the flood control purpose.

During the drawdown period, the objective of operation is to maximize hydropower production for a given total storage amount vis-à-vis the demands. Recall that a reservoir system can generate the maximum amount of power when all the reservoirs are full because the hydraulic heads will be the highest in this case. If the available water in the system is limited, it should be allocated among the reservoirs in such a way that the hydropower production is maximized. The governing variables here are storages, inflows, installed capacities and efficiencies of power plants. Note that in a small reservoir, the rate of increases of head per unit volume of additional water is higher as compared to a large reservoir (with more surface area), all other things remaining the same. As shown in Fig. 11.7, the volume of water needed to increase the head by 1 unit in a smaller reservoir (V1) is less than the volume needed (V2) to increase the head in a larger reservoir by the same increment.

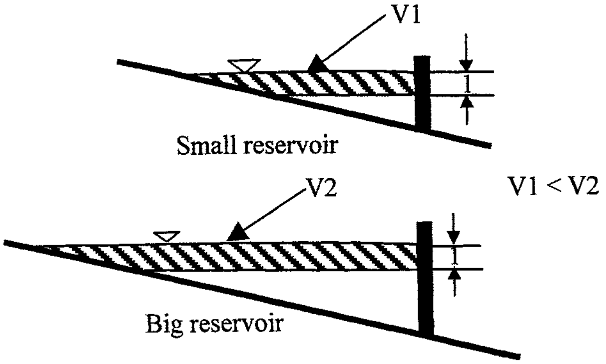

Fig. 11.7 Volumes of water needed for a unit change in head in a big and small reservoir [adapted from Lund and Guzman (1998)].

The important variables in hydropower production, namely, reservoir capacity, volume of inflows, and efficiency of power plant, determine the overall suitability and ranking of the reservoirs. Thus, if the storage can be increased, it should be in reservoirs with the greatest ability to produce power and vice versa. Following this reasoning, Lund and Guzman (1998) suggested a measure that can be used to rank the reservoirs:

$$V_i = a_i \eta_i (\sum_{j=1}^{i} I_j) \tag{11.2}$$

where V_i is the increased power production per unit increase in the storage, a_i is unit change in hydropower head per unit change in storage, and η_i is the power generation efficiency, all for reservoir *i;* and I_j is direct inflows and releases into reservoir *j*; and the summation is for all reservoirs upstream of reservoir *i*. The most upstream reservoir in the series is numbered 1 and so on. The reservoirs are ranked according to the V_i values. The filling begins at the highest value of V_i and proceeds in the descending order.

A variant of this rule is the *Storage Effectiveness Index* method developed by the U. S. Army Corps of Engineers (USACE, 1985).

Storage Effectiveness Index Method

This method was developed by USACE to maximize firm hydropower production during the drawdown season. For each reservoir, a storage effectiveness index is calculated for each time-step, using forecast inflows and power demands for the current time-step and remaining time-steps in the drawdown season. Lund and Guzman (1998) gave the following computational steps of this method:

Step 1. Find the firm energy requirement for the current time-step E_f.
Step 2. Estimate the shortfall of firm hydropower production due to insufficient inflows to the system.

$$S_f = E_f - 720 \sum_{i=1}^{n} I_{Ui} H_i(S_i) \eta_i \tag{11.3}$$

where S_f is the energy shortage (kW-hr) for the current time-step (a month consisting of 720 hours), I_{Ui} is the inflow upstream of reservoir *i* during the current time-step (m^3/s), and H_i is the hydropower head (m) for reservoir *i* as a function of current reservoir storage S_i. The eq. (11.3) assumes that all flows can be passed through turbines to generate power.

Step 3. For each reservoir, the volume of water required for that reservoir to individually eliminate the shortfall (ΔS_i) is estimated as

$$\Delta S_i = S_f / (720^* H_i \eta_i) \tag{11.4}$$

where H_i is the average head at which the volume ΔS_i is released.

Step 4. For each reservoir, the energy loss in the remainder of the drawdown season due to the release of ΔS_i during this time-step is estimated.

Step 5. The storage effectiveness ratio (SER) for reservoir *i* is

$$SER_i = E_{Li} / S_f \tag{11.5}$$

where E_{Li} is the drawdown season power loss due to drawdown of reservoir *i* by ΔS_i units.

This ratio is calculated for each reservoir and the reservoirs with the lowest ratios are to be drawn down first.

11.5.3 Reservoirs in Parallel

The simplest configuration of parallel reservoirs is shown in Fig. 11.8. It consists of two reservoirs located on two different streams which join downstream of the reservoirs. The direct demands of reservoirs 1 and 2 are D_1 and D_2, respectively. Either or both of the reservoirs can meet demand D3. An important difference in operation of series and parallel reservoirs is that the release from an upstream reservoir cannot be captured by a downstream reservoir. Therefore, balancing of the operation is important in such cases. Rules to operate parallel reservoirs are summarized in Table 11.2.

In this configuration, the commonly followed procedure is to discharge water first from the reservoir with larger drainage area or potential inflows per unit storage capacity. To that end, the drainage areas to storage volume capacity ratios for two reservoirs are compared (assuming the runoff per unit of drainage area is the same). The reservoir with the larger ratio will supply water for demand D_3 before the other reservoir is drawn down. Discharging water first from the reservoir having the largest drainage to the storage volume capacity ratio will usually result in a reasonable conservation of water.

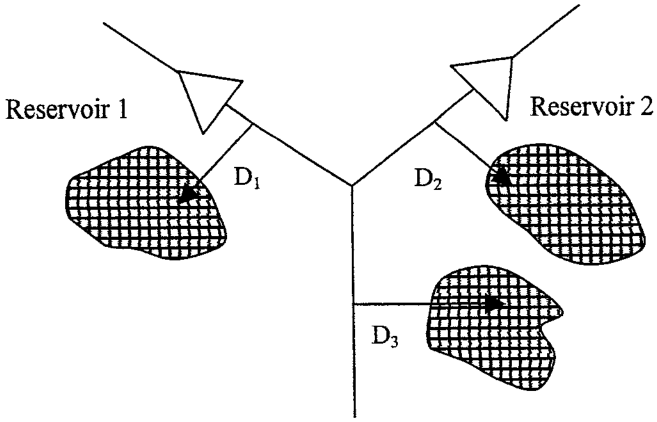

Fig. 11.8 A system of two parallel reservoirs.

Several types of rules have been developed for conservation operation of parallel reservoirs during refill periods. A typical objective of such system is to minimize expected shortages. These rules prescribe ideal releases or storage levels for reservoirs in parallel to avoid the inefficient condition of having some reservoirs full and spilling, whereas other reservoirs have unused storage capacity. The severity of shortages is reduced by minimizing uncontrolled spills from the system.

Table 11.2 General rules operation of reservoirs in parallel [adapted from Lund and Guzman (1998)].

Purpose	Refill period	Drawdown period
Water supply	Equalize probability of spill among reservoirs	Equalize probability of emptying among reservoirs
Flood control	Keep more vacant space in reservoirs likely to receive bigger floods	Not applicable
Energy storage	Equalize EV of energy spill among reservoirs	For the last time-step, equalize EV of seasonal energy spill among reservoirs
Hydropower production	Maximize storage in reservoirs with greatest energy production per unit of water	Maximize storage in reservoirs with greatest energy production per unit of water
Recreation	Equalize marginal recreation improvement of additional storage among reservoirs	Equalize marginal recreation improvement of additional storage among reservoirs

Note: EV = expected value.

New York City Rules

The New York City (NYC) rules are helpful for a parallel system of reservoirs that have a single demand downstream of all reservoirs. Clark first stated these rules in 1950 for the NYC water supply system. In operating this system, an attempt was made to have the storage in each of the watersheds, at all times, fall on the same percentage year. The reservoirs are operated to minimize expected shortages by using the probability of spills. Clark noted that the physical spill is minimized when the probabilities of spill at the end of the refill season are the same for each reservoir. The deficit in meeting the target is minimized when the physical spill is minimum.

Application of the NYC rule requires prediction of inflows. Evidently, better accuracy of predictions would result in lesser spillage of water. A high degree of accuracy in predicted inflows is not critical in the early periods of the refill season. The releases are recalculated at each period and the reliability of flow forecasts becomes progressively more important as one reaches the end of the refill season. The NYC rule has been found to be optimal or near-optimal in a variety of operating conditions and system configurations. The optimality of rules also depends on the coefficient of variation of mean monthly flows and the correlation between flows on adjacent streams.

Originally, NYC rules assumed that the unit value of water is the same in each reservoir. But this may not be always the case. In a water supply system, the quality of raw water affects its unit value because poor quality water would entail higher treatment costs. For hydropower generation, a unit volume of water in a reservoir having greater head can generate more energy and will, therefore, have higher value. The NYC rules were later modified to handle situations where the unit value of water varies between reservoirs but is constant in any individual reservoir.

The general form of the NYC rule equates the probabilities of spill at the end of the refill season adjusted by the unit value of water for each reservoir

$$h_i \Pr[\, CQ_i \geq K_i - S_{fi} \,] = \lambda, \text{ for all } i \tag{11.6}$$

where h_i is the unit value of water in reservoir i, CQ_i is the cumulative inflow to reservoir i from the end of the current period to the end of the refill season, K_i is the storage capacity of reservoir i (assumed to be the same in every period), S_{fi} is the storage at the end of the current period for reservoir i, and λ is constant across all reservoirs in parallel. Note that if the unit value of water is the same among reservoirs that are supplying water, h_i can be incorporated into constant λ in eq. (11.6).

Generally, historical data are used to estimate cumulative inflows CQ_i. The release from each reservoir for the current period is found by knowing the initial storage, expected inflow and the end-of-period storage that satisfies eq. (11.6). The sum of these releases should equal the total downstream target release. A trial-and-error procedure may have to be adopted to find the releases. If the purpose of reservoirs is energy generation, the probabilities of spill of the potential energy are equated.

Hydropower Rules

For steady-state hydropower production at reservoirs in parallel, the storage effectiveness of parallel reservoir j can be defined as

$$V_j = \eta_j a_j I_j \tag{11.7}$$

where subscript j refers to an individual parallel reservoir. While emptying parallel reservoirs sequentially, those with the smallest V_j are emptied first. The filling should take place in the reverse order. Sheer (1986) and Lund and Guzman (1998) have derived rules for complex cases.

11.5.4 Other Rules

Some other useful rules for reservoir operation are discussed in this section.

Space Rule

The space rule seeks to leave more space in reservoirs where greater inflows are expected, or where inflows with greater potential energy are expected in the case of energy storage. The space rule, proposed by Maass et al. (1962), seeks to minimize the volume of spills. An inefficient condition arises in operation of multiple reservoirs when some of them are full and spilling, and others are unfilled. The spill-minimizing objective implies that this rule is especially suitable in the system's refill season. This requires monitoring storage volumes and estimating future inflows. When parallel reservoirs are operated by this rule, the objective is to equalize the probability that the reservoirs will have filled at the end of the drawdown refill cycle. In that event, all the reservoirs will be full and spilling, full and not spilling, or partly full, the unoccupied storage space being proportioned to inflows during the drawdown refill cycle. Mathematically, the rule is:

$$\frac{S_{\max\,j} - S_{Ijk} - Q_{jk} + R_{jk}}{\sum_{j}^{m} (S_{\max\,j} - S_{Ijk} - Q_{jk}) + R_T} = \frac{Q_{j,n-k}}{\sum_{j}^{m} Q_{j,n-k}} \tag{11.8}$$

where $S_{\max j}$ is the full capacity of the jth in a series of m parallel reservoirs; S_{Ijk} is the initial contents of the jth reservoir in the kth month of a series of n months; Q_{jk} is the flow into the jth reservoir in the kth month; R_{jk} is the release from the jth reservoir in the kth month; R_T is the sum total of target releases required; and $Q_{j,n-k}$ is the predicted flow into the jth reservoir for the remaining $n-k$ months of the drawdown refill cycle. Solving the above equation for R_{jk}, the release from the jth reservoir in the kth month is:

$$R_{jk} = \left[\sum_{j}^{m} \left(S_{\max\,j} - S_{Ijk} - Q_{jk}\right) + R_T\right] \times \left(Q_{j,n-k} / \sum_{j}^{m} Q_{j,n-k}\right) + S_{Ijk} + Q_{jk} - S_{\max\,j} \tag{11.9}$$

subject to the constraint

$$0 < R_{jk} < (S_{Ijk} + Q_{jk}) \tag{11.10}$$

The space rule is useful in situations where inflow forecasting is reliable as in the case of runoff from snowmelt. For other types of streamflows, the effectiveness of the space rule would be a function of the coefficient of variation of the mean monthly flows, the correlation between flows on the adjacent stream and the reliability of flow forecasts. Although this rule aims at minimizing the water spilled in the system during the remainder of the drawdown-refill cycle, it does not guarantee minimum system spill. The rule also does not provide the total system yield R_T which has to be computed separately. ReVelle (1999) has proposed a few variants of the space rule

The following numerical example illustrates the use of space rule to determine releases from two parallel reservoirs.

Example 11.1: While operating two parallel reservoirs for irrigation for a given month, the target output is $400*10^3$ m³. The unregulated flow from the catchment below the reservoirs and above the point of irrigation diversion is nil. For the data given in Table 11.3, find the release from each reservoir.

Solution: The computations following eq. (11.9) are shown in Table 11.3. To meet the target output of $400*10^3$ m³, the release from the reservoir number 1 would be $36.4*10^3$ m³ and from the reservoir number 2, it will be $363.8*10^3$ m³. The releases from the individual reservoirs are determined based on the storage in them and the expected inflows.

The space rule can be modified and used to apportion releases among reservoirs for flood control, based on short intervals of time. It is also valid when each unit of water is

Table 11.3 Determination of reservoir releases using space rule (Example 11.1).

Row	Details	Reservoir No. 1 (10^3 m^3)	Reservoir No. 2 (10^3 m^3)	Total for reservoir 1 and 2
A	Maximum storage capacity	200.0	2000.0	2200.0
B	Storage at the beginning of month	100.0	1000.0	1100.0
C	Empty storage space at the beginning of month, *Row A – Row B*	100.0	1000.0	1100.0
D	Inflow during the month	50.0	500.0	550.0
E	Target irrigation release for the month	To be computed		400.0
F	Total space that would be available at the end of current month $C - D + E$	To be computed		950.0
F	Predicted inflow between end of current month and end of refill cycle	100.0	1000.0	1100.0
H	Fraction of required space at end of month	100/1100	1000/1100	1
I	Allocation of space at the end of month = $F * H$	86.4	863.6	950.0
J	Storage contents at end of month $A - I$	113.6	1136.4	1250.0
K	Release for the current month $B + D - J$	36.4	363.6	400.0

of equal value in a given reservoir but not in different reservoirs. However, the space rule must be modified in its form to deal with this situation. Unequal values of water may be present in a system, for instance, when there is a power-plant downstream of reservoir B to generate electric energy from irrigation releases but no such plant exists below reservoir A. Space rule can also be applied to maximize the economic value of the system output by minimizing spills of the higher valued water from reservoir B at the expense of spills of downstream valued water from reservoir A. This rule provides flexibility in operation by increasing release towards the end of the drawdown-refill cycle to free reservoir space for predicted inflows that might otherwise spill.

Pack Rule

The pack rule uses streamflow forecasts during the last few months of drawdown-refill cycle and tries to avoid spills by additional releases of water in advance (say, for secondary energy generation). The rule was so named by Maass et al. (1962) because the expected future spill is as tightly packed as possible into future spare turbine capacity. Mathematically,

$$R_d = Q_{n\text{-}k} - (S_{max} - S_{Tk}) - P_{n\text{-}k} \tag{11.11}$$

where R_d denotes the additional releases for the current month *k* for the generation of dump energy; $Q_{n\text{-}k}$ is the predicted flow into the reservoir for the remaining *n-k* months of the drawdown refill cycle; S_{max} is the full reservoir capacity; S_{Tk} is the reservoir contents in the current month after current flows have been added and releases made to meet the target output for energy, and P_{n-k} is the useful water capacity of turbines for the remaining *n-k* months of the drawdown refill cycle. If the right hand side of the equation is not positive, $R_d=0$ and the equation is further subject to constraint: $P_c \geq R_d \leq S_{Tk}$ where P_c is the useful water capacity of turbines in the current month after releases have been made through turbines to meet the target output for energy.

The pack rule can be applied whenever releases beyond the specified output requirements are of value. In Fig. 11.9, the operation of a reservoir which also generates hydropower is depicted. Assume that the drawdown-refill cycle begins in the month of January and ends in December. By applying the pack rule, secondary energy was generated during the last two months of the cycle, shown by rectangles with brick shaped hatching. In the last month of this drawdown refill cycle, there is spill of water of the order of $25*10^6$ m^3, shown by dotted rectangle. If the pack rule is applied, additional secondary electric energy, shown by rectangles hatched with slanting lines, can be generated during the months of September and October. Clearly, the application of the pack rule has two advantages. It minimizes spill of water which has no value and may as well cause some unwanted consequences in the downstream areas. At the same time, additional secondary energy is generated by the system to increases the overall benefit.

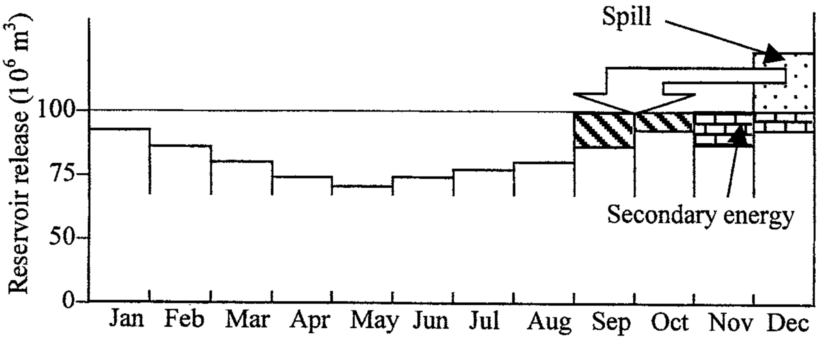

Fig. 11.9 Operation of a reservoir. Shown here is spill that would result from a rigid rule and generation of extra secondary energy using this spill by applying pack rule.

Hedging Rule

The aim of hedging is to distribute the anticipated shortage uniformly so that its severity is reduced. It is sometimes economical to accept a small current deficit in releases so as to decrease the probability of more severe water or energy shortage at a later date. The effect of hedging is brought out in Fig. 11.10 for a reservoir operating for irrigation/water supply. In Fig. 11.10(a), the total volume of release is 240 million m^3 and the volume of deficit is 55 million m^3. The largest deficit of 30 million m^3 occurred in the month of January followed by a deficit of 20 million m^3 in February. This deficit was distributed by following the hedging rule and the revised release schedule is shown in Fig. 11.10(b). After the released were revised, the largest deficit is 15 million m^3 which occurred in the months of January and February. This severity is just half of the severity of the first case and is expected to be less damaging although the total volume of deficit stays the same.

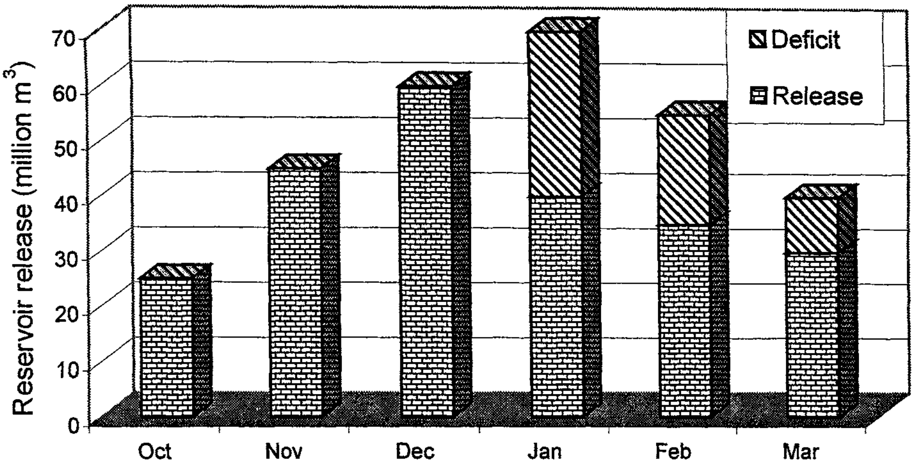

Fig. 11.10(a) Hedging rule – monthly outputs and shortages without hedging.

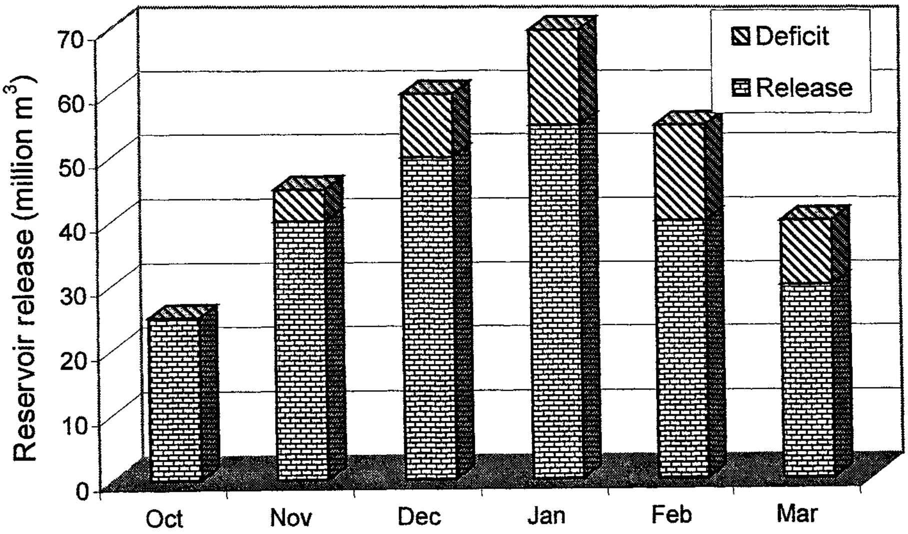

Fig. 11.10(b) Hedging rule – monthly outputs and shortages with hedging.

The economic justification of the hedging rule arises from nonlinear benefit or loss functions of the proposed uses of water. If the marginal values of water for specific uses are constant, the economic losses from shortages will be linear. It is well known that the streamflows are stochastic and to some extent the demands, particularly those arising from agriculture areas, too are stochastic. These two facts will suggest that it is better to postpone shortages as long as possible. However, the marginal values of water for a specific use are not constant. Since large deficits are more damaging than smaller ones, is preferable to a avoid the possibility of suffering heavy deficits.

11.5.5 Selective Withdrawal

The quality of supplied water is very important for many uses. For example, cold water is not good for temperature sensitive crops, such as rice. The temperature and dissolved oxygen content of water released from a dam affect biological life in the stream. Due to taste and odor problems, it is not desirable to supply poor quality water for domestic use. Since the quality of water stored in a reservoir varies with depth, in some instances it is required to supply water from a particular layer. This mode of operation is termed as selective withdrawal. For this purpose, an intake that can withdraw water from the desired layer(s) is a pre-requisite. Many reservoirs have conduits to allow rapid release of turbid water during flood events. Fontane et al. (1981) presented a methodology combining optimization and simulation for determining operational guidelines for selective withdrawal structures to meet downstream water temperature objectives.

11.6 RESERVOIR OPERATION FOR FLOOD CONTROL

Among the measures of flood control, a storage reservoir with gates to control the outflow is perhaps the most effective means. The moderation of a flood through storage is achieved by storing a part of flood volume in the rising phase of the hydrograph and releasing the same gradually in the receding phase of the flood. The degree of moderation or flood attenuation depends on the empty storage space available in the reservoir when the flood impinges on it. The flood control pool must be emptied as quickly as the downstream flooding conditions allow; this will reduce the risk of highly damaging future releases, should a major flood occur in quick succession.

The reservoir regulation consists of storing peak flows over and above the safe (non-damaging) carrying capacity of the channel at the damage point in the reservoir. The reservoir is emptied after the passage of the flood to make space for control of subsequent floods. In Fig. 11.11, ABCDE represents the inflow hydrograph. The line ZZ represents the non-damaging carrying capacity of the river channel downstream of the reservoir. From point B to point D, the natural flow in the river exceeds its safe carrying capacity. If there were no reservoir, from the time corresponding to point B up to point D, the flood water will have spilled over the channel banks and cause damage. The moderated release from the reservoir under the ideal operation is given by the dotted curve AGDF. As soon as the inflow begins to increase, the release is gradually increased till some point G where the release equals the safe carrying capacity. While the inflows from point B to D exceed the safe carrying capacity of the downstream channel, the release is maintained within safe range by storing the volume in the segment BCD in the flood control zone of the reservoir. After point D, the inflows continue to fall rapidly but the release, while still in safe zone, exceeds inflow so that the reservoir is quickly emptied.

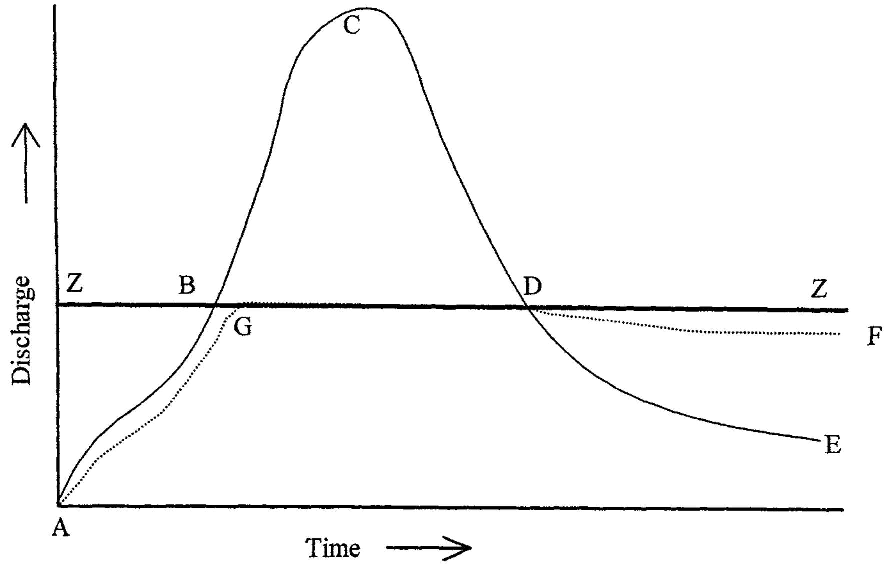

Fig. 11.11 Ideal operation of a reservoir for flood control.

The above operation is an ideal situation which is possible only if the perfect foreknowledge of the hydrograph is available. In absence of such information, the release curve may deviate from the ideal shape. For example, if the operator makes smaller releases in the early part of the hydrograph, it is likely that the reservoir will completely fill before point D. In that eventuality, the operator will be forced to make releases in excess of the safe carrying capacity of the downstream channel thereby causing flood damages. Conversely, if at the beginning of a flood event, the operator starts making higher releases in the expectation of a major flood and such a flood does not occur, the reservoir may not fill to the desired level by the end of the filling season.

The efficiency function of a flood control reservoir can be defined by (Nagy et al., 2002):

$$K = f(Q_d^{max}) \tag{11.12}$$

where K represents the storage capacity to ensures that the maximum outflow Q_d^{max} from the reservoir will be lower than the maximum upstream flood. At the starting point of the efficiency function curve is a reservoir with zero capacity which will not modify the inflows. At the other end is a reservoir whose capacity is equal to the volume of the entire flood flow (V):

$$K = V, \text{ and } Q_d^{max} = 0 \tag{11.13}$$

Nagy et al. (2002) have described a method to determine the required capacity of a storage reservoir using the efficiency function and a given reliability.

Before discussing the approaches for flood control operation of a reservoir, a technique that is used to allocate storage space for flood control in a multi-purpose reservoir is discussed.

11.6.1 Flood Control Reservation Diagram

To allocate storage space in a multi-purpose reservoir, the flood control diagram is especially useful. The curves of this diagram define the amount of vacant storage space to be kept available to control the floods of known or expected magnitude over a specified time interval. In calculating the amount of space required, it is assumed that releases in excess of channel capacity will not be made.

The vacant space required for flood control in a reservoir on any date is the difference in the volume of inflow between the date in question and the date of maximum storage (end of filling season) and the volume of water released from the reservoir. Usually the time step size of the analysis is a week or a fortnight. To derive curves for the first day of the various time steps during the filling season, the historical flow record is routed through the reservoir. The amount of storage space required to control runoff after the date in question is plotted for each year. The positive values indicate the vacant space that must be kept available in addition to the minimum reservation for flood control. The years having negative values of required space indicate that the flood could have been controlled with less than the minimum reservation for flood control. A line is then fitted through the points for the date in question.

The flood control space requirement can be linearly related with the remaining season runoff. The generalized equation is:

$$Y = mX + C_t \tag{11.14}$$

where Y is the required space in volume units, m is the tangent of the straight line, X is the remaining season runoff in volume unit, and C_t is the ordinate intercept (volume units) which is a function of the time (day of the filling season). If the flood control space given by eq. (11.14) is not available in the reservoir on the given date, one must make sufficient releases so that enough vacant space is created. Depending on the inflows, this release may or may not exceed the normal outflow required to meet the conservation demands.

It is important to note that the flood control reservation chart does not indicate the size of the release to be made. The decision concerning the rate of release will depend on the actual inflows. If the flood control space required is violated marginally and the inflows are not likely to increase rapidly for some time, the release may be made to gradually vacate the encroached space. This will save water, particularly if subsequent runoff is small. If, however, the inflows are likely to be large, the reservoir should be drawn down rapidly.

As the filling season progresses, the runoff likely to be generated between the current time and the end of filling season decreases and thereby, the space required for flood control also decreases. It implies that the reservoir should be gradually filled so that the objective of having the reservoir full at the end of the monsoon season can be met. In other words, the amount of joint storage space committed to flood control decreases while that for conservation increases as the season progresses. At the end of the flood season, the entire joint-use storage space is committed to conservation.

11.6.2 Approaches to Reservoir Operation during Floods

Two approaches are common in controlling flood peaks by reservoirs. The first consists of operating the reservoir to reduce every flood peak by the maximum possible amount. In the second approach, attention is focussed on moderation of larger floods; the smaller flood peaks are not given much attention. In any case, it would be ideal that the reservoir is operated such that the release is always less than the safe carrying capacity of the downstream channel. Since substantial water may be temporarily stored in the reservoir, the water level rises above the FRL. After the flood has peaked, the reservoir is gradually bought back to FRL.

There can be three possible approaches for regulating an incoming flood. This classification is based on the peak of the incoming flood.

Regulation Based on the Maximum Use of the Available Space in Each Flood Event

This type of regulation aims at reducing damaging stages at locations sought to be protected as much as possible during each flood with the judicious use of the available space. The possibility of having an appreciable portion of the flood control storage capacity already filled up before the occurrence of a large subsequent flood is disregarded.

The regulation to obtain the maximum benefits during ordinary floods can be successful depending on the ability to forecast flow conditions at the reservoir and flood-prone areas below. This requires adequate network of hydrologic stations and capability to properly evaluate weather forecasts. This method is helpful in cases where the available flood control storage is insufficient to control larger floods which occur less frequently. The disadvantage is that the available protection is limited if a dangerous flood occurs after most of the available capacity has been utilized to regulate lesser floods.

Regulation Based on the Control of Project Design Flood

In a project which has a flood regulation capacity based on the control of the project design flood, the regulation may be based on the assumption that each incoming flood might develop as design flood. The release rates are so established that all the flood control storage capacity is utilized if the current flood turns out to be the same as the project design flood.

As the project design flood occurrence is an unusual event, a schedule for its regulation will normally afford a satisfactory moderation of most floods. However, with this strategy, a less satisfactory regulation of lesser floods (which occur more frequently) can result at times.

Regulation with Combination Method

The best overall plan of operation often results by adopting a combination of the above two methods. For instance, to protect an agricultural area, a regulation plan for the maximum damage reduction during the main farming season may be desired but it may not be the most advantageous when the fields are empty. As the flood waters carry nutrients etc., there is a view that controlled flooding of agricultural areas may not be all that bad. Nevertheless, it is desirable to reserve the reservoir storage to provide assured flood protection to an important town or areas (e.g., an industrial estate). In such cases, a schedule of releases to assure greater control of major floods at the expense of less regulation of moderate floods may be more desirable.

11.6.3 Pre-depletion of Reservoirs

In some situations, it is not desirable to allow the reservoir level to rise above FRL for some reasons, e.g., the land near the periphery could not be acquired. In that situation, the outlet capacity should be large enough to control the rise of the reservoir water level. Alternatively, the reservoir level is lowered before the arrival of the flood by making anticipatory pre-releases and the level is brought back to FRL after the flood peak has passed (see Fig. 11.12). This strategy involves the use of inflow forecasts and the confidence of the operator in making pre-releases, and therefore, depends on the reliability and timely availability of forecasts. This aspect is further discussed in Section 11.8.

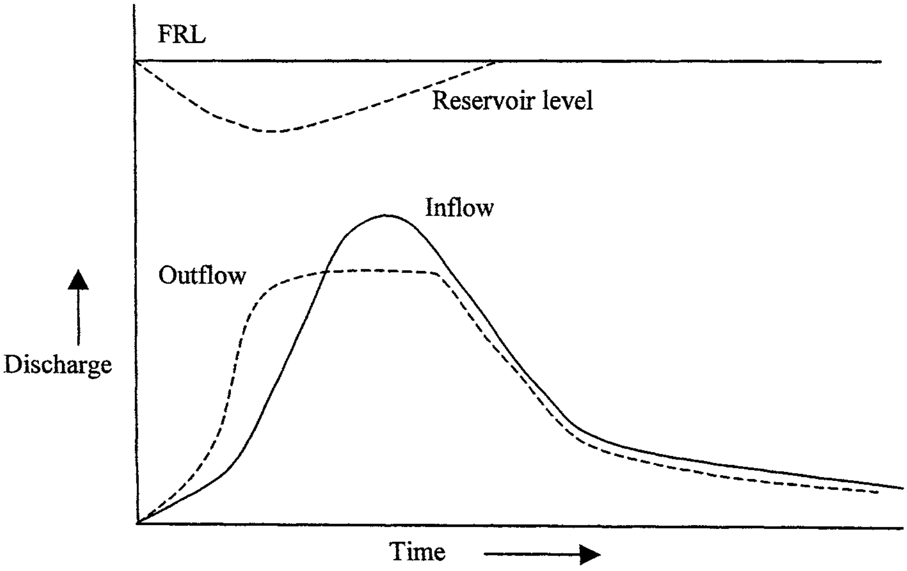

Fig. 11.12 Flood moderation through reservoir pre-depletion.

In some reservoirs, the space available for flood control is small compared to the volume of the design flood. In such situations also, pre-depletion proves to be useful for flood moderation. Consider that the reservoir is at or below FRL at the beginning of the flood control operation. The operator can lower the reservoir level by pre-releases to create the storage space for flood moderation. During the passage of the flood (see Fig. 11.13), the reservoir is regulated such that the maximum water level is below MWL. Thereafter, releases are made such that the reservoir level comes back to the desired elevation. Thus, the flood is moderated using the space created by pre-releases and the space between FRL and MWL. If not constrained by other factors, this approach perhaps involves making the most efficient use of the reservoir storage space.

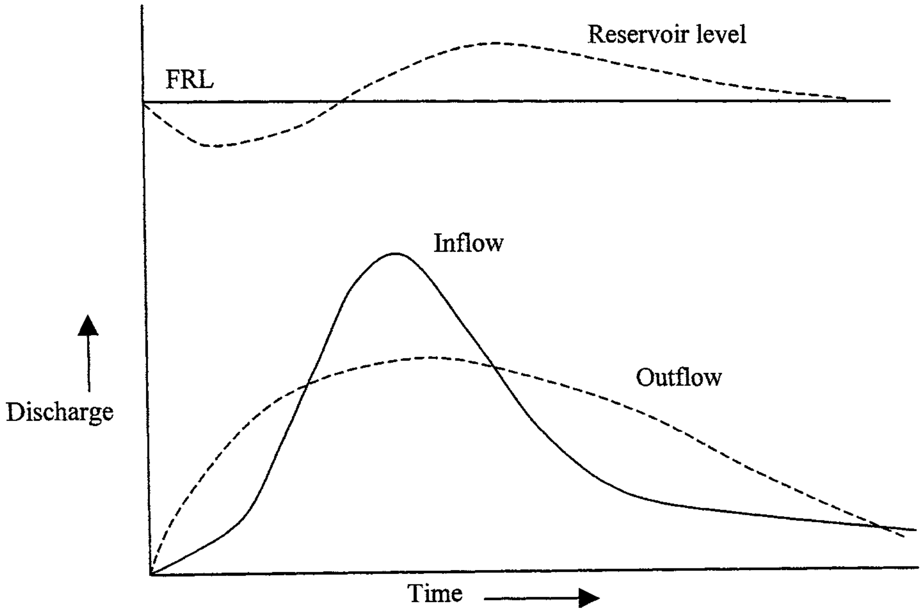

Fig. 11.13 Flood moderation through pre-depletion and use of flood control space.

The important considerations in pre-depletion are discussed below.

When the pre-depletion should commence?

The pre-depletion should begin as soon as the (expected) inflows exceed a specified value. This value depends on the reservoir and catchment characteristics. A typical value could be about 10% of the minimum of the outlet capacity at FRL and the safe carrying capacity of the downstream channel.

How much the reservoir level should be depleted?

The extent of pre-depletion will depend on the current reservoir level, the shape of elevation-capacity curve, the inflow and their rate of increase, and the rainfall in the catchment. The net rainfall in the catchment provides an estimate of the volume of inflows expected in the reservoir. If the reservoir is pre-depleted by this much volume, one can be reasonably sure to refill the reservoir back to the original level.

At what stage after passage of flood, should the reservoir be filled back?

It would be necessary to have an estimate of the net rainfall in the catchment at each time step (usually 1-3 hour). This would give the volume of expected inflows. After the passage of peak, the reservoir re-filling should commence whenever the expected inflows are a little more than the volume required to re-fill the reservoir up to the desired level.

11.6.4 Normal and Emergency Operation

In many cases, the current hydrologic situation in terms of observed values of relevant meteorologic variables, streamflows, etc. are not available at the reservoir control centers. In such situations, the decision making process should be based on indicators which the operator at the dam site can easily observe. These are: a) the current reservoir level and its rate of rise, and b) the inflows and their trend (whether these are increasing or decreasing).

The operation of a reservoir for flood control usually begins as soon as the water level in the reservoir exceeds some specified level, such as FRL, or the inflow rate is greater than a threshold. The operation policy for flood control regulation could be based on the concept of a cut-off reservoir-level, cut-off inflow magnitude and the nature of inflow (i.e., rising or falling). While developing the regulation policy, it is assumed that the operator has the following objectives:

a) to attain FRL at the earliest and to maintain it;
b) to control the rise of reservoir level above FRL to the extent possible;
c) to utilize the reservoir storage capacity fully before releasing water at rates exceeding the safe channel capacity; and
d) to release water at rates above the non-damaging discharge in the downstream reaches for the least possible time.

A procedure for operation of flood control reservoirs that is based on two variables has been found to be satisfactory in most cases. The first variable, the *reservoir critical level (RCL)* is a level between FRL and MWL. If, during the passage of a flood wave, the reservoir level rises above RCL, it indicates that the normal operation procedure has not been able to contain floods within the desired range. This could be due to limitations of storage capacity or outlet release capacity or because the magnitude of the flood is higher than anticipated. As a very high water level in the reservoir can endanger the safety of the dam, it is necessary that the reservoir level be brought down below RCL at the earliest. To do that, the operator may be forced to make releases which may cause damages in the downstream reaches. The *critical flood inflow (CFI)* may also be defined in a similar way and indicates abnormally high inflows to the reservoir. If the actual inflows are greater than CFI at any stage, and the reservoir level is above RCL, the emergency operation schedule should be followed without delay.

The RCL and CFI depend on the reservoir capacity, particularly between FRL and MWL, the capacity of spillway and other outlet works, safe carrying capacity of downstream channel, the volume under the design flood hydrograph and the peak of design flood. No simple analytical technique is available to determine RCL and CFI for a reservoir. These can be determined by simulation through a trial and error approach. As a rough guide, the initial value of RCL may be taken as half way between FRL and MWL and CFI equal to the safe carrying capacity of the downstream channel. Simulation of the operation of the reservoir is then carried out with either the design flood hydrograph or the hydrograph of a major flood that has occurred in the past. The detailed working table should be examined to flag the periods during which the reservoir level was undesirably high and the releases were more than the safe carrying capacity of the downstream channel. The parameters RCL and CFI are now tuned to best attain the stated objectives. Normally, convergence is rapidly achieved. In case, a system of reservoirs is analyzed, this procedure should be carried out first for individual reservoirs. After these parameters have been obtained for each reservoir, the entire system is studied. At this stage, the parameters for individual reservoirs may have to be changed a little bit so that the overall performance of the system is the best.

Now, the reservoir operation scenario can be classified into two categories: *Normal Operation* and *Emergency Operation.*

Normal Operation

The normal operation of the reservoir for flood control begins as soon as the first signs of an impending flood event are noticed. Specifically, the normal operation policy is initiated when

a) the reservoir level is between FRL and RCL and the inflow rate is less than CFI; or
b) the reservoir level is between FRL and RCL, the inflow rate is greater than CFI, and it is decreasing; or
c) the reservoir level is greater than RCL, the inflow rate is less than CFI, and it is decreasing.

Under normal operation, water is released at a rate which is less than or equal to the safe carrying capacity of the downstream channel. The aim is to bring the reservoir back to FRL at the earliest so that the next flood, if any, may be moderated.

Emergency Operation

This mode of operation is invoked when the flood build-up is bigger than anticipated and the normal operation has failed to control it. Either the reservoir level is already high or the inflows are big, indicating the likelihood of an extreme flood. Under the emergency operation, the safety of a dam becomes important. The emergency operation policy is followed when

a) the reservoir level is greater than RCL and the inflow rate is less than CFI but it is increasing; or

b) the reservoir level is greater than RCL, inflow rate is greater than CFI, and it is decreasing; or
c) at any reservoir level, inflow rate is more than CFI and it is increasing.

If the release at the rate of safe downstream channel capacity is likely to cause overtopping, the release is made equal to the outlet capacity at the current elevation. The objective is to bring the reservoir down to safer level at the earliest and ensure safety of the dam. The minimum rate at which water should be released from the reservoir under emergency conditions is the lesser of the inflow rate and the outlet capacity. A software on the above lines was reported by Jain and Goel (1999).

11.6.5 Flood Control Operation of a Multi-Reservoir System

A reservoir system may have single-purpose reservoirs in series, in parallel, or a mixed configuration of multi-purpose reservoirs.

Flood Control Reservoirs in Series

Since the objective of flood control is to transmit the water from the system at the earliest while causing the least damage, the operation policy is opposite to the conservation operation policy. Therefore, in a series of reservoirs serving solely for flood control, any flow which does not cause flood damage should be allowed to pass at the earliest. If the excess water is to be stored, the headwater reservoirs should be filled first and the downstream reservoirs are to be emptied first. Since the closest upstream reservoir to a location to be protected has the greatest flood control capability and most of the area to be protected lies in the lower reaches of a basin, this approach appears to be the best strategy for flood management.

Flood Control Reservoirs in Parallel

The usual approach for flood control operation of parallel reservoirs is to maintain the reservoirs in balance in terms of occupied capacities and flood flow from respective catchment areas. These reservoirs are operated for flood control such that the combined release plus the flow from the intermediate catchment is below the safe carrying capacity of the channel at the damage center and if this is not possible, the flow in excess of the safe channel capacity is exceeded by the smallest amount and for the least possible time. If the releases are to be increased, the increment is higher for the reservoir whose flood control storage occupation is the most, or which is likely to receive higher inflow. The situation is reverse when the releases are to be curtailed.

The objective behind balancing of parallel reservoirs is to logically and properly use the volume of flood control storage available, while maximizing non-damaging releases from the system. The principle of balancing the flood control storage on parallel reservoirs could be developed on the same lines as the space rule.

The USACE procedure to allocate flood control space between two parallel reservoirs (see Fig. 11.8) with a common downstream flood damage center is as follows:

1. Route the reservoir design flood or other observed major floods for reservoir 1 while making releases at maximum non-damaging rate. Allow reservoir 2 to make the remaining releases up to the maximum non-damaging level. Plot the space required in reservoir 1 versus the total space required.
2. Perform the same analysis in reservoir 2. Plot the space required at reservoir 2 versus the total space required.
3. The ratio for balancing flood storage between the two reservoirs should lie between these two curves.

Multi-purpose Reservoirs

Permanent allocation of space exclusively for flood control at the top of the conservation pool becomes necessary in those multipurpose reservoirs for which flood control is one of the main purposes and floods can be experienced at any time of the year. The size of flood reservation may vary according to the magnitude of expected floods. The flood storage space allocation at different times of the year is so determined that incoming floods would be absorbed or mitigated to the maximum degree. The floods should be so regulated that the downstream damage does not exceed permissible limits even if a big flood comes. If the reservoir is in a region where floods are experienced only in a particular season, the allocation of space for flood management varies during different parts of the flood season, depending on the magnitude of likely floods. After the flood season is over, this space may be used to store water for other uses.

A multi-purpose reservoir whose primary objective is flood protection should be operated to provide maximum moderation to potentially dangerous floods. Water can be stored in the joint use space of a reservoir for other purposes as long as the flood control operation is not hindered. Before the onset of a flood, the main decision demanding the operator's attention is the amount of flood control space that ought to be available at any time and the size of the releases required to create additional space, if necessary.

The reservoir operation should also include the periodic assessment of future incoming volumes based on rainfall and other data gathered from the raingage stations in the catchment. The frequency of such a review depends on the catchment size and storm characteristics. The release decisions are also influenced by intuition, experience, and judgment of the operator.

Example 11.2: This example illustrates the rule curves for Panchet dam, India. Several reservoirs were constructed in the Damodar valley (India) in the 1950s for the purpose of flood control, irrigation, power generation, and municipal and industrial water supply. The major dams are Tilaiya, Konark, Maithon and Panchet. Consider, for illustration, the Panchet dam whose drainage area is 10961 sq. km and the average annual runoff is 4541 Mm^3. The Panchet dam has a gross storage capacity of 1476 Mm^3 at an elevation 135.67 metre (445 ft). The top of the conservation pool is at an elevation 125 metre (410 ft) at which the storage capacity is 392 Mm^3. Thus, the storage space of 1084 Mm^3 is available for flood control. The maximum observed flood is 8558 cumec while the spillway design flood is 17853 cumec. The installed capacity of power plant is 40MW. The rule curves for Panchet multipurpose reservoir are shown in Fig. 11.14.

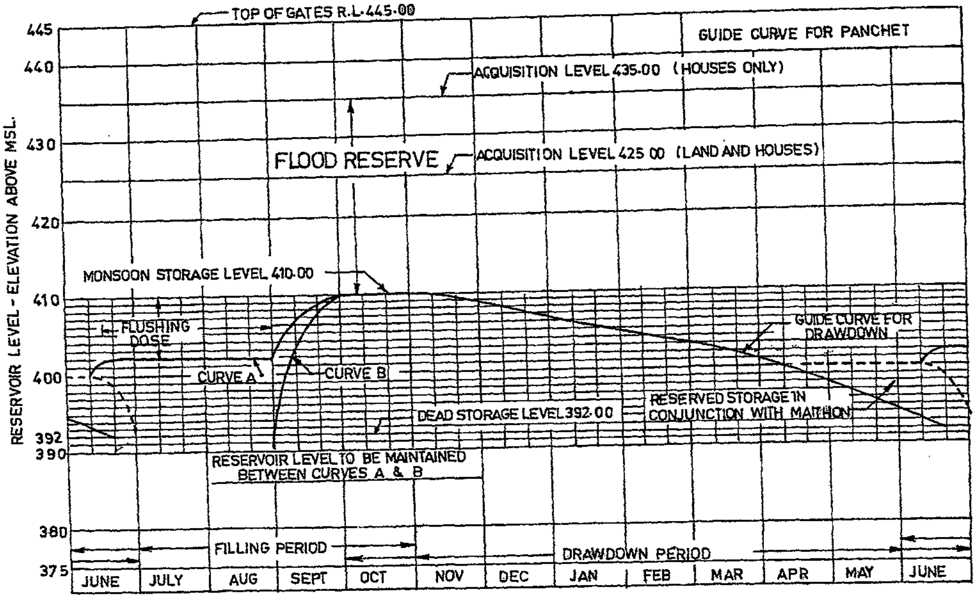

Fig. 11.14 Rule curve for Panchet reservoir, Damodar valley (India).

According to the guidelines, the reservoir level may be drawn down to 119.5 m to meet the power requirements in June. On the first day of July, the Panchet reservoir should be kept as near an elevation 121.95 metre (400 ft) as possible to ensure generation of hydropower. The reservoir level should be kept between curves A and B (Fig. 11.14) during July, August, and closer to curve A during September. The rule curves are to be followed such that the reservoir attains a level of 125 m on 1st October.

Whenever the water level rises above 125 m, flood control operation commences and this operation ceases as soon as the level comes down to 125 m. This reservoir along with another reservoir, namely Maithon, is operated to control floods at a downstream point by trying to keep the maximum flow of less than 7084 cumec at that point. The emergency operation for flood control is triggered when half of the flood control space in the reservoir is occupied. The outflow from the dam is made equal to inflow when 100% flood control space is occupied.

The water available from monsoon storage and dry season flows are utilised to meet the committed requirements of irrigation, industrial and municipal water supply and hydropower needs during the drawdown season. The reservoir operation guidelines prepared by the Damodar Valley Corporation also recommend that before the commencement of monsoon, inspection and operation of gates should be made for smooth operation during monsoon. The communication system should also be checked for trouble-free service.

11.7 SYSTEM ENGINEERING FOR RESERVOIR MANAGEMENT

Determination of the reservoir operation policy to efficiently manage available water is a complex problem because it involves random hydrologic events. Many attempts have been made to solve this problem using optimization and simulation models. Reservoir optimization models allow the user to generate operating decisions that are optimal in some "measurable" sense. By making a number of runs of a simulation model with alternative decision policies, a (near) optimal solution can be reached. Maass et al. (1962), Loucks e al. (1981), Wurbs (1996) and many others have discussed optimization and simulation models and their underlying differences at length. Wurbs (1996) has presented an annotated bibliography of optimization and simulation models.

11.7.1 Optimization

The Linear Programming (LP) and Dynamic Programming (DP) optimization techniques have been extensively used in water resources. Loucks et al. (1981) have illustrated applications of LP, Non-linear programming (NLP), and DP to water resources. Many reviews of applications of systems techniques for water resources problems have been published from time to time, such as those by Yakowitz (1982), Yeh (1985), Simonovic (1992), and Wurbs (1993).

After little modifications, the model for design of a reservoir that was presented in Section 10.10.4 can be used to determine an operation policy. The modified formulation is

$$\text{Max } D \qquad (11.15)$$

$$S_t + I_t - R_t = S_{t+1} \quad t = 1, 2, \ldots n \qquad (11.16)$$

$$R_t \geq \alpha_i D, \quad t = 1, 2, \ldots n \qquad (11.17)$$

$$S_t \leq C, \quad t = 1, 2, \ldots n \qquad (11.18)$$

$$S_t, C, D, R_t \geq 0, \quad t = 1, 2, \ldots n \qquad (11.19)$$

This formulation does not consider evaporation loss from the reservoir which could be significant in arid climates. An approximate method to include evaporation losses involves establishing a linear relation between the average reservoir storage for the period and surface area A_t:

$$A_t = a + b(S_t + S_{t-1})/2 \qquad (11.20$$

The volume of water lost during the period (m^3) will be the product of area and the depth of evaporation (d_t). After incorporating evaporation loss, the constraint given by eq (11.16) becomes

$$S_t + I_t - R_t - a^*d_t - b^*d_t^*S_t/2 - b^*d_t^*S_{t-1}/2 = S_{t+1}, \quad t = 1, 2, \ldots n \qquad (11.21$$

Note that in eq. (11.21), R_t includes release as well as spill. The generation of hydropower is also considered in a similar manner. The average head (h_t) in period *t* is expressed as a function of the average storage

$$h_t = c + d(S_t + S_{t-1})/2 \qquad (11.22)$$

The power generated during the period is [see eq. (10.6)]

$$P_t = 9.817\, T\eta\, R_t\, [c + d(S_t + S_{t-1})/2] \qquad (11.23)$$

or

$$P_t = \alpha\, R_t\, c + \alpha\, R_t\, d\, S_t/2 + \alpha\, R_t\, d\, S_{t-1}/2 \qquad (11.23a)$$

where $\alpha = 9.817\, T\eta$. This equation can be used with the formulation given by eq. (11.15) to (11.19). It is well known that LP cannot be directly used to determine the maximum firm power. The model has to be repeatedly solved due to the non-linear nature of eq. (11.23). The basic model described here can be extended to a system of reservoirs. ReVelle (1999) has described many such formulations for a variety of purposes.

Young (1967) was the first to propose the use of a linear regression procedure to derive general operating rules from deterministic optimization. The approach that he had used is called “Monte Carlo DP”. Basically his method is to generate, for the river in question, a number of series of synthetic annual stream flow sequences using the Monte Carlo technique. For each of these, he used a DP formulation with a forward computation procedure. The optimum policies obtained for each of the synthetic stream flow sequences were then used in regression analysis in an attempt to determine the causal factor influencing the optimal policy. This method appears to be sound and is computationally efficient. The results are a good approximation of the true optimal policy.

A programming model for design of multi-reservoir flood control system was developed by Windsor (1975). Karamouz and Houck (1987) derived general operating rules using deterministic DP and regression (DPR). The DPR model incorporates a multiple linear regression procedure suggested by Bhaskar and Whitlach (1980). Rules for operation of multireservoir systems can also be developed by SDP which requires an explicit characterization of streamflow probabilities and a loss function. This approach has been used by Butcher (1971), Loucks et al. (1981), and numerous others. SDP approaches to deriving operating rules suffer from large computational needs for large problems. Attempts to reduce computational requirements by increasing the coarseness of storage and flow discretizations lead to approximate results. A comparison of SDP with other operating rules by Karamouz and Houck (1987) and Johnson et al. (1991) show that well-designed conventional operating rules may perform equally well or sometimes better than SDP.

Optimization models are frequently used in reservoir operation studies employing flow forecast as input. Datta and Burges (1984) derived a short-term operation policy for multipurpose reservoirs from an optimization model with the objective of minimizing short-term losses. They examined the sensitivity of various performance criteria for operation of a single reservoir to the accuracy of forecast streamflow volumes. The study revealed that when there is a trade-off incurring one unit of storage deviation and one unit of release deviation from respective target values, the optimized solution depends on uncertain future streamflow as well as the shape of the loss function.

The application of optimization models for multiple-reservoir operation is bit cumbersome. Difficulties in application include model development, trained manpower, cost of solution, an adequate inclusion of uncertain future hydrologic conditions, inability to identify and quantify all relevant objectives, and the need for better interaction with the user. However, this should not be interpreted as a judgment against optimization techniques which are very useful tools for water resources analyst. Another approach which is being used these days to account for stochasticity of inflows is the fuzzy logic programming. The fuzzy set theory was introduced by Zadeh (1965). In a fuzzy set of objects, there is no sharp boundary between those elements that belong to the class and those that do not. The membership function assigns a grade of membership, varying from zero to one to each object. Jairaj and Vedula (2000) applied this approach to multireservoir optimization.

11.7.2 Simulation

Since it is not possible to do experiments with a real reservoir, mathematical simulation models are developed and used in studies. Experiments can be conducted using these models to provide insights into the problem. For reservoir operation, the model studies bring into focus certain aspects of operation which serve to improve the manager's ability to control the system wisely. The simulation models associated with reservoir operation include the mass-balance computation of reservoir inflows, outflows, and changes in storage. They may also provide an economic evaluation of flood damages, hydropower benefits, irrigation benefits, and other similar characteristics. The simulation technique has provided a bridge from early analytical tools for analyses of reservoir systems to complex general-purpose packages. According to Simonovic (1992), the concepts inherent in simulation are easier to understand and communicate than other modeling concepts.

Simulation models can provide more realistic and detailed representation of reservoir systems and their operations (such as detailed responses of individual reservoirs and channels or the effects of certain time-varying phenomena). They also allow added flexibility in deriving responses which cannot always be readily defined in economic terms (recreational benefits, preservation of fish and wildlife, etc.). The time required to prepare inputs, run models, and other computational demands of simulation are much lower than those of optimization models. The simulation results will readily bring out the trade-offs in case of multiple objectives. The ability is helpful in arriving at the best policy of system operation. Practical real-time operation requires the specification of reservoir operating rules which is much simpler in simulation.

A number of general-purpose computer software are available which can be used to do analysis related to planning, design, and operation of reservoirs. Most of the software can be run on microcomputers which are widely available these days. Moreover, once the data required for particular software has been prepared for a problem, it is easy to modify the same and hence the consequences of various alternative design/operation decisions can be quickly evaluated.

Examples of simulation date back to the early 1950s. Perhaps, the first contributions were made by the Harvard Water Program (Maass et al., 1962). Probably one of the most popular and widely used generalized reservoir system simulation models is the HEC-5 model developed by the Hydrologic Engineering Center (Feldman, 1981; Wurbs 1996). Some other well-known simulation models are: the Acres model (Sigvaldson, 1976); the Streamflow Synthesis and Reservoir Regulation (SSARR) Model (USACE, 1987), the Interactive River System Simulation (IRIS) model (Loucks et al, 1989) and the Water Rights Analysis Package (WRAP) (Wurbs et al., 1993). Lund and Ferriera (1996) studied the Missouri River reservoir system and found simulation models to be superior to classical regression techniques for inferring and refining operating rules derived from deterministic DP. Jain and Goel (1996) have presented a generalized simulation model for conservation operation of a reservoir system based on rule curves. Despite the availability of several generalized models, the need to develop simulation models for specific reservoir system is usually present as each reservoir system has some unique features.

Multi-reservoir simulation models used to assess the impact of various operating policies are useful only if the voluminous output from all the various runs can be compared and evaluated. The analyst computes the means and variances and the time distribution of reservoir performance indicators, such as reservoir storage volumes, releases, associated benefits or losses, and these can be used for policy evaluation and comparison. The evaluation can also make use of concepts, such as system reliability, resilience, and vulnerability [see Chapter 8]. Simulation models for reservoir operation are of value for aiding in assessing possible impacts of alternative operating policies and for forecasting the future state of the system, given a specific operating policy and predicted hydrologic scenarios.

Steps for Application of a Simulation Model

The steps to perform a simulation study of a reservoir system are as follows:

a) Prepare the diagram of the system showing names of reservoirs and diversion weirs/barrages, their location and the length and direction of rivers and tributaries.
b) Collect general details about the operation like the number of control locations in the system, initial month, day and hour, and the total number of periods of operation.
c) Assign numbers to all the control points (storage reservoir, diversion weir, barrage etc.) starting from the upstream node. Some models may require special numbering system.
d) Collect general details about each location which include maximum capacity up to the full reservoir level, initial storage, elevation-area-capacity table, demands, minimum release to be made in the downstream channel, and the evaporation depths for all the months of the year.
e) For each structure, calculate the local flow coming from the free catchment area at that structure for all the periods of operation. If routing is to be performed, the values of the required parameters are to be provided.
f) Simulate the operation of the system. An examination of the working tables of the various reservoirs, performance statistics, and graphs of relevant variables, such as release, reservoir storage, and demand, will show the improvement that can be made in

the operation policy.

g) Modify the input operation policy and run the model again. Repeat this procedure till the desired results are obtained.

The performance statistics that are typically used in step (f) include time and volume reliabilities, the largest shortage and spill, the frequency of spill, the maximum number of periods of consecutive shortages.

11.7.3 Network Flow Models

The network flow models have been discussed in Chapter 5. Most models developed for water resources systems use the "out-of-kilter" algorithm (OKA) to find the optimum flow in the network. This algorithm solves a special class of LP problems, each of which can be represented as a "capacitated network", i.e., as a series of nodes and interconnecting arcs. The objective is expressed as the minimum collective cost of flows through all arcs, subject to two types of constraints. The first type is simply the mass balance equation at each node. The second type of constraints state that every arc flow must be within some specified lower and upper limits. Fortunately, many water resource problems can be transposed into an equivalent network representation. Storage changes in reservoirs during individual time periods and changes in system operation through a sequence of time periods can also be represented effectively. It is also easy to assign priorities to various uses of water. An attractive feature of the OKA is its computational efficiency. The *Surface Water Allocation Model AL-V* developed by Martin (1981) uses OKA for optimal water allocation.

11.7.4 Linear Decision Rule

A linear decision rule (LDR) relates releases from a reservoir to storage content and decision parameters. The concept of a decision rule in reservoir planning and operation was proposed by Young (1967) who found that LDRs provided as good or better fit to stochastic input data as more complicated rules when quadratic loss functions are associated with release deficits. Extensive work on LDR in water resource management was published in the 1970s in a series of papers beginning with ReVelle et al. (1969). This rule is very useful in formulations that involve chance-constraints. In the simplest form for reservoir management, LDR assumes that the release is a linear function of reservoir storage:

$$R_t = S_t - b_t \tag{11.24}$$

in which R_t denotes the release during time period t, S_t denotes the storage at the beginning of time period t, and b_t is the decision parameter that is determined by optimizing a criterion function. Note that reservoir losses are not explicitly considered herein. Numerous investigators have modified, extended, or applied this rule to a variety of reservoir optimization problems. Loucks and Dorfman (1975) suggested the following LDR for the initial reservoir storage in period $t+1$:

$$S_{t+1} = \lambda_t I_t + b_t \tag{11.25}$$

in which I_t is the inflow during period *t*, and λ_t is a known coefficient ($0 \le \lambda_t \le 1$). When this decision rule is substituted in the storage continuity equation, the LDR for reservoir releases in each within-year period *t* is obtained:

$$R_t = S_t + (1 - \lambda_t)I_t - b_t \tag{11.26}$$

In this rule, coefficient λ_t can be interpreted as a weight given to inflow in the current period while determining the release. As λ_t is increased from 0 to 1, the rule becomes more conservative and more reservoir capacity is required to meet commitments. The LDR by ReVelle et al. (1969) is a special case of eq. (11.26) when λ_t equals 1:

$$R_t = S_t - b_t = I_{t-1} + b_{t-1} - b_t \tag{11.27}$$

If a reservoir is operated by following this decision rule, the releases at the beginning of a time period can only be determined if $0 \le b_t \le S_t$ since the initial storage S_t will be known at the beginning of period *t*. If $b_t < 0$ or if $b_t > S_t$, then the release will be determined in part by the current inflow I_t, which will not be known at the beginning of period *t* (Loucks and Dorfman, 1975).

A less conservative decision rule is obtained when λ_t is equal to 0 in eq. (11.26):

$$R_t = S_t + I_t - b_t = I_t + b_{t-1} - b_t \tag{11.28}$$

and

$$S_{t+1} = b_t$$

This rule equates b_t with the random end of the period storage volume and thus does not allow a release commitment at the beginning of the period. In eq. (11.27) that is obtained by setting λ_t equal to 1, release is a function of the past inflow I_{t-1} while in eq. (11.28) where $\lambda_t = 0$, release depends on the current inflow I_t. ReVelle and Gundelach (1975) proposed an LDR that is a function of current storage and past inflows:

$$R_t = S_{t-1} + \beta_t I_t - \beta_{t-1} I_{t-1} + \ldots + \beta_{t-k} I_{t-k} + b_t \tag{11.29}$$

in which constants β_t, β_{t-1}, β_{t-k} are to be determined. This extension considers incorporation of the stochastic nature of the streamflow process into an LDR formulation. This rule results in smaller reservoirs than the original LDR with an objective of minimizing the reservoir storage capacity subject to several performance criteria (Gundelach and ReVelle, 1975).

LDR has been a subject of considerable controversy and is said to produce conservative results. The conservative nature of LDR originates from (Joeres et al., 1981): (1) inappropriate comparisons of the multi-objective the LDR model with a single purpose "yield maximizing design methods," (2) inappropriate comparisons of non-comparable operating modes, and (3) the assumption in LDR that streamflows in successive periods are independent of each other. Loucks and Dorfman (1975) mentioned that the conservative nature of LDR arises because the rule itself is an additional operating constraint in the system. They suggest that while this technique may be suitable for screening studies, it is not satisfactory to derive optimal operating policies for reservoir(s). ReVelle (1999) has provided many extensions of the basic LP model given in Section 11.7.1 for the allocation of reservoir services among water supply, flood control, and hydropower using chance-constrained programming (discussed in Section 4.6.1). For example, the chance-constrained model to determine the maximum firm release R_F from a reservoir will be:

$$\text{Max } R_F \tag{11.30}$$

$$R_F + S_{t+1} - S_t + L_t = I_t \qquad t = 1, 2, \ldots n \tag{11.31}$$

$$R_F - S_t \le I^{\alpha}_t \qquad t = 1, 2, \ldots n \tag{11.32}$$

$$S_n \ge S_0 \qquad t = 1, 2, \ldots n \tag{11.33}$$

$$S_t \le C, \qquad t = 1, 2, \ldots n \tag{11.34}$$

$$S_t, C, R_F, L_t \ge 0, \quad t = 1, 2, \ldots n \tag{11.35}$$

where L_t is spill from the reservoir during period t. The constraint of eq. (11.32), which is similar to constraint of eq. (4.78), ensures that storage at the end of period t is greater than or equal to zero with probability α. I^{α}_t is the inflow for the month t that is exceeded with a probability α, also known as the $(1-\alpha)$- percentile flow for month t (see Fig. 11.15). The constraint of eq. (11.33) ensures that the storage at the end of the computation horizon is not greater than the beginning storage or the total volume of release should not exceed the total inflows. Note that the stochasticity of inflows can be taken into account either through the chance-constrained models or through synthetic inflow sequences. For correct answers, it is necessary that the synthetic inflows properly reproduce the critical flows of the record or the persistence of low flows is maintained. In the chance-constrained programming, these low flows are accounted for through I^{α}_t.

11.7.5 Recomposition-Decomposition Approach

In large river basins where many developmental activities may be going on, it may be cumbersome to carry out the systems study in one mathematical operation. A system has various components and each of them has a different function to perform. Therefore, it can be expected that the models for the various components could be different. In other words, there is no single universal model governing all components and functions.

A meaningful study of such a system can be carried out by decomposing it into smaller sub-systems, each of them being within manageable limits for systems analysis. Each subsystem can be optimized from a functional point of view. Towards the end, these sub-systems can be recomposed into the whole system. This approach has many advantages.

Hall and Dracup (1970) proposed that a water resources system could be decomposed into 5 sub-system on the basis of purposes, viz., Watershed Sub-system (WSSS), River Regulation Sub-system (RRSS), Water Distribution Sub-system (WDSS), Water Use Sub System (WUSS), and Water Waste Regulation Sub-system (WWRSS). The WSSS is normally managed for purposes other than water resources. This sub-system usually affects the flow of the water in rivers and sediment into reservoirs. The RRSS consists of reservoir storages (both surface reservoir and ground water reservoirs for the purpose of converting the stochastic flow into assured and regulated flow. WDSS consists

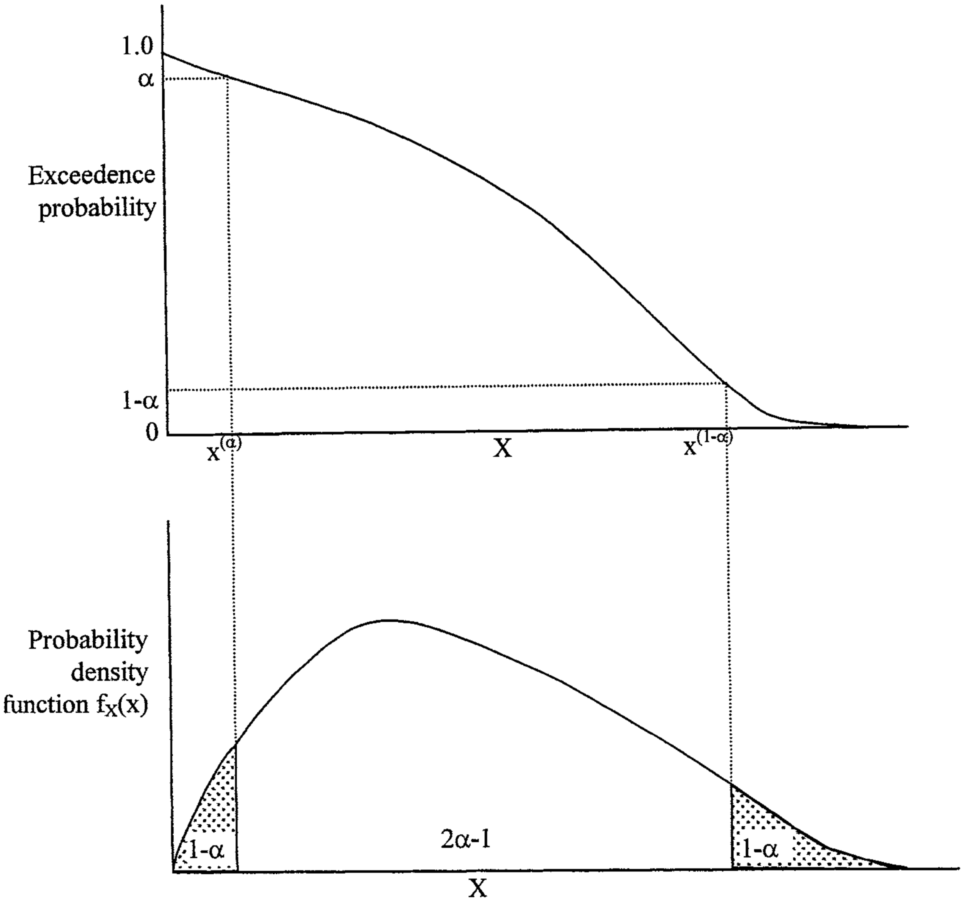

Fig. 11.15 Probability distributions of random variable X. The probability of exceedence of $x^{(\alpha)}$ is α.

of river reaches and canal systems. This starts from a terminus of RRSS to a place of the use of water. WUSS comprises the fields in the command receiving water from the distribution system. WWRSS consists of all collection and treatment works involved in removing waste or unused water from WUSS and for treating it to the extent required for ultimate disposal or reuse.

A somewhat similar strategy that has been proposed in the literature is referred to as "linear dynamic decomposition programming". In this approach, DP is used for optimizing individual reservoirs and LP for combining the reservoirs collectively into an integrated optimization model. The approach uses dual variables from the LP solution to constrain the DP solution. In turn, the DP solution provides column vectors for the LP matrix. An optimal solution is obtained after a series of iterations back and forth between the LP and DP portions of the model.

A multi-reservoir system may have many small reservoirs whose ability to influence the system operation and the impact on the overall performance may be marginal. Many times, the desired data about these small reservoirs are not available. In studies dealing with such systems, it is common to aggregate some of the reservoirs to reduce computational or data demands of mathematical models. Usually, reservoirs in series are easier to aggregate and disaggregate than reservoirs in parallel.

Example 11.3: This example illustrates application of optimization for reservoir operation through a real-life case.

The Upper Indravati project is situated in the Nowrangpur and Kalahandi districts of Orissa, India. The reservoir has a catchment area of 2630 km^2 and a gross storage capacity of 2300 Mm^3. The objectives of this project are hydropower generation and irrigation. The installed capacity of the hydropower plant is 600 MW. After power generation, flow from the tailrace channel will be diverted into irrigation canals by a weir. The canal would irrigate 128,000 ha of agricultural land. A distinct feature of the project is trans-basin diversion of water from the River Indravati (Godavari basin) into the River Hati (Mahanadi basin) for power generation and subsequent irrigation. The operation policy for this reservoir was derived using optimization.

Monthly inflows into the reservoir for a period of 32 years from 1951-82, were obtained from Orissa Irrigation Department. The monthly evaporation losses from the reservoir and the irrigation demands determined in an earlier study were considered. The reservoir dead storage and gross storage capacity are 814.5 and 2300 Mm^3, respectively. To compute the head for the hydropower generation, a constant tail water elevation of 265.00 m and an average head loss of 12 m was considered. A turbine efficiency of 92.0 % and generator efficiency of 97.5 % were assumed.

Development of Reservoir Operation Policy

The DP approach was used to determine optimal reservoir releases. The objective function (OF) of the DP model was to maximize energy production, subject to typical system constraints. This will, however, increase the likelihood of spill and there is a trade-off between the two aspects of the operation. The OF, initially chosen, was a weighted sum of hydropower generation and saving water in storage:

$$Max\sum_{t=1}^{N}\left(\frac{H_t * rel_t}{10^6}+\frac{Sav_t}{10^4}\right) \tag{11.36}$$

where H_t represents the average effective head (m) during time period t (average pool elevation - tail water level), rel_t is the release (Mm^3) during period t, N is the number of time periods within the optimization horizon, and Sav_t is the average storage for the period t (Mm^3).

Constraints

Two types of constraints were considered: those which represent the inherent system characteristics and will not change during optimization and others which are loss or penalty functions. These constraints are discussed below.

i. Mass-balance or continuity equation

$$S_{t+1} = S_t + \mathrm{inf}_t - \mathrm{elos}_t - \mathrm{rel}_t - \mathrm{spil}_t \qquad t = 1,2,\ldots,N \qquad (11.37)$$

where S_t is the initial reservoir storage, inf_t is the reservoir inflow, elos_t is the evaporation loss, and spil_t is the spill from the reservoir, all for time period *t*.

ii. Storage S_t can vary only between the maximum and minimum storage bounds:

$$S_{min} \leq S_t \leq S_{max} \qquad t = 1,2,\ldots,N \qquad (11.38)$$

iii. Hydro-power generation is proportional to the release and the operating head and is given by

$$\mathrm{gen}_t = c * 9.8 * \mathrm{rel}_t * H_t * \eta \qquad t = 1,2,\ldots,N \qquad (11.39)$$

where gen_t is the energy generated in period *t*, *c* is a constant for converting release in MCM to cumec, and η is the efficiency of power plant.

iv. The hydropower generation is limited by the installed capacity:

$$\mathrm{gen}_t \leq \text{Installed capacity}, \qquad t = 1,2,\ldots,N \qquad (11.40)$$

v. The penalty for not meeting irrigation demands:

$$\mathrm{ben}_t = -\{\mathrm{rel}_t - \mathrm{dem}_t\}^2 / 10^8 \qquad t = 1,2,\ldots,N \qquad (11.41)$$

where ben_t is the benefit during time period *t*, and dem_t is the demand during time period *t*.

The problem is one of adopting the appropriate objective function and identifying a suitable loss function which can accommodate both the objectives, viz., maximization of power and minimisation of irrigation deficit. This was finalized after a trial and error procedure by first choosing a set of functions, then running the DP model and finally evaluating the model performance as per certain criteria and by repeating the entire procedure after altering the functions. The Discrete Differential DP (DDDP) approach was used to determine the optimal releases.

Sensitivity Analysis of the Objective Function

As the hydropower generation is a function of head as well as discharge, a term for higher storage (i.e., higher pool elevation) was added in some runs. A penalty was also imposed for deficits. Several combinations of OF and penalty were examined. The criteria adopted for the performance appraisal of various DP models were: total generation during the optimization period, irrigation deficit and spill. A summary of the results obtained from some competing model runs is given in Table 11.4. From this table, the following inferences can be drawn about the Indravati system:

In runs 1 and 2, only penalty function is different. For case 2, spill is less, power generation is about the same but deficit is very large. The penalty function forced releases in the current period if water was available and this caused large deficits in latter periods.

Comparison of runs 1, 2, and 3 shows that the penalty function of run 3 is more appropriate as the spill is least, generation is maximum and the deficit is least, although OF is the same. The term for storage in the objective function of run 4 has more weight than has run 5. This results in higher amounts of spill and reduction in power generation, whereas the deficit remains unaffected. The OF of runs 5 and 6 is the same but the penalty function for run 6 is dependent on head also. In run 6, the spill and deficit are least and power generation is the maximum.

Table 11.4 DP performance appraisal for alternative objectives and penalty functions.

SN	Objective Function	Penalty Function	Spill Mm^3	Generation (MW)	Deficit ($M\ m^3$)	Mean Square Deficit ($M\ m^3$)
1	$(S_{av} + H_{av} * rel)/10^6$	$-(rel - dem)^2/10^{11}$	3524	100376	2590	4737
2	$(S_{av} + H_{av} * rel)/ 10^6$	If water is available & rel<dem: $-ben- [H_{av} * (rel-dem)^2]/10^6$ else $-(rel-dem)^2/10^{11}$	2512	100969	5253	9081
3	$(S_{av} + H_{av} * rel)/ 10^6$	$- ben - H_{av} * (rel-dem)^2/ 10^6$	2265	101217	2309	2484
4	$S_{av}/10^4 + (H_{av} * rel)/10^6$	$-(rel - dem)^2/10^8$	3795	99641	2326	2363
5	$(H_{av} * rel)/10^6$	$-(rel - dem)^2/10^8$	3607	99914	2326	2362
6	$(H_{av} * rel)/10^6$	$- ben - H_{av} * (rel-dem)^2/ 10^6$	2242	101248	2309	2484

Overall, the set-up of run 6 appears to be the best. Regarding DDDP, it was observed that by increasing the number of iterations and successively reducing the corridor width, the total deficit is distributed among a larger number of time periods with smaller deficits. Further details of this study are available in Jain et al. (1999).

11.8 REAL-TIME RESERVOIR OPERATION

Generally, the reservoir operation policy is developed taking into account the demands of the past and using data from historical or synthetic time series of hydrological variables. But the probability that an actual event will occur in the same way as prior events of the same type is small. A reservoir system can be efficiently operated if the time interval between the occurrence of an event and the execution of the control adapted for that event is short. In real-time operation, the release decisions are based on short-term information.

The definition of short-term varies in accordance with the purpose. If the reservoir is operated for flood control, the short term may refer to (multi) hourly operation and if it is serving conservation purposes, the short term may be a day or longer.

The term *real-time control* denotes the execution of a decision process concurrently with a physical system such that the results of the analysis based on on-line data are available in time to usefully control the physical system. Here on-line implies that the data about the system are received without any delay as the events take place and are immediately used. In real-time reservoir operation, the release decisions for a finite future time horizon are taken, based on the condition of the reservoir at that instant when these decisions are to be taken and the forecast about the likely inflows/demands over this time horizon, if available. After a certain time interval, new information about the reservoir state becomes available, the forecasts are updated, and the decisions are modified in light of these.

Real-time operation is especially suitable during floods where the catchment response changes rapidly and decisions have to be taken quickly and adapted frequently. A model of the system is developed in which release is a decision variable. A forecasting algorithm is used to provide inflow forecast for a finite number of future time periods based on the present state of the system as well as its past behavior. Using these, a mathematical model is used to determine the optimum amount of water to be released from the reservoir. Although the optimum releases are determined for a finite number of future time periods, they are implemented only for the immediately next time period. After this period, the next set of observations becomes available and the entire process is repeated. The control process can also use the information about river flows at critical locations while taking a decision. This process of control of a system is known as adaptive control – the decision is adapted based on the feedback received from the system (Fig. 11.16).

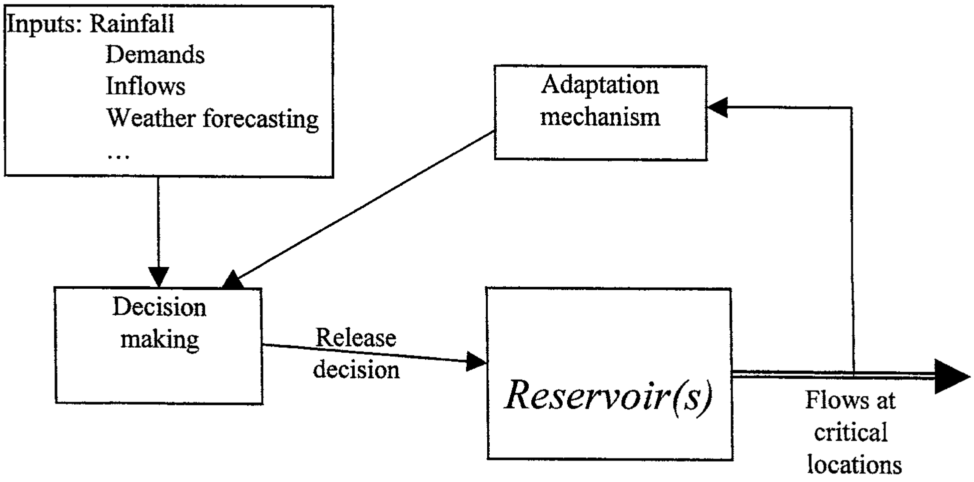

Fig. 11.16 Adaptive feedback control of a system.

11.8.1 Real-time Hydrological Forecasting

Hydrological forecasting is an important application of hydrology. The benefits from real-time operation of a reservoir can be substantially increased if good forecasts are available. The main components involved in the inflow forecasting are weather forecasting, rainfall-runoff modeling, and channel routing. For a forecast in proper time, the transmission of information and processing of data has to be done rapidly by employing computers.

Real-time flood forecasting involves estimation of discharges in a river some period prior to its occurrence. The forecast lead-time proves useful in mitigating some of the adverse effects of flooding. The forecast lead-time depends on catchment characteristics which affect the time taken by the catchment to transform rainfall into discharge at a point. The utility of a forecast depends on the accuracy and timeliness. Clearly, under-prediction of a flood event can lead to a dangerous situation and may result in loss of life and damage to property in the affected areas. On the other hand, over-prediction of a flood event will result in avoidable and unnecessary high releases of water from the dam, evacuation of people from the downstream area likely to be affected by floods, unnecessary flood fighting measures, and panic among the people.

A forecast would be valuable if available much before the event takes place. The forecast lead-time is the advance time prior to the occurrence of the event over which the forecast is issued. In general, the longer the forecast lead-time, the greater will be its utility. However, the errors in forecasts increase progressively with increase in the lead-time. Thus, there is a need for proper balance between the accuracy and the lead-time, although a certain minimum lead-time (depending on the local conditions) is necessary to organize flood fighting measures. Normally, a lead-time of at least 10 to 12 hours is necessary to organize a meaningful flood fighting measure.

The field of hydrologic forecasting is vast and highly developed. A detailed discussion is beyond the scope of this book and a large volume of reference material is available, for example, Lettenmaier and Wood (1993).

11.8.2 Special Considerations in Real-time Operation

The necessity for real-time operation arises from the fact that the inflows to reservoirs are random in nature and uncertain. To know the actual inflows to the reservoir and to forecast future inflows, adequate data collection and transmission network is required. During periods of high flows, operation of gates becomes an important aspect and needs to be given due consideration. In case, where it may not be possible to fully absorb the flood in the reservoir, efficient information dissemination system is required to warn the people downstream well in advance.

Data-acquisition System: A sophisticated hydrologic model and a fast computer system serve their useful purpose only if the data acquisition system is reliable and fast. The exchange of data between widely separated data observing equipment and forecasting center is called telemetry. In this system, usually no human element is involved and this completely eliminates the human error and reduces the time of observation and transmission of data. Chapter 2 provides a detailed discussion on data acquisition and processing. A successful application of the real-time operation requires a good telemetry system.

Downstream Conditions: A reservoir may have other reservoirs downstream of it, an area to be protected immediately downstream of it, or an area to be protected far away from it. In the latter case, there will be a large uncontrolled catchment between the reservoir and the damage centers. While making release from the reservoir, it will be important to ensure that the peaks of the flow from the uncontrolled area and the reservoir release do not occur at the same time at the damage center.

The nondamaging channel capacity at major damage centers downstream of the dam is a primary consideration in sizing the flood control storage and regulation. Usually, the nondamaging discharge tends to reduce with time. Before the commencement of flood season, a survey of the downstream area must be made to ascertain any changes in the channel carrying capacity. If there are significant changes in this capacity, these will have to be incorporated in the operation procedure.

Regulation of Outlet Structures: The outlet capacity is an important factor in flood control operation of a reservoir. This capacity is crucial in cases where inflows expected could be more than the outlet capacity because in such cases the safety of the dam itself may be under threat. It is not only the total outlet capacity but the capacity at various levels of a reservoir that influences the operation decisions. There may be large capacity sluices and crest spillway with or without gates. The sluices which can operate at lower levels of reservoir are effective in operation where pre-releases from the dam are required to be made. However, the discharging capacity of sluices varies with the square root of head and they are not very effective at higher heads. Spillways are more effective in disposing flood waters because the discharge varies with head raised to the power of 1.5.

The spillway gates can be operated manually or mechanically. In the manual operation, the gate movement is slow. The gate operation in mechanical system is rapid and a uniform rate of opening can be maintained. If there are rapid variations of inflow or if some emergency situation arises, gates may have to be operated frequently and the time taken for opening/closing of gates becomes important. In case of mechanical operation, stand-by arrangements must be made for supply of electric power. When operating outlets, the rate of change in release and the reservoir level should be within permissible limits.

Upstream Conditions: Backwater surface profiles must be studied to determine acquisition and easement requirements upstream of the reservoir. Sedimentation deposition will adversely affect these areas and this must be anticipated and evaluated. Upstream conditions determine the levels up to which a reservoir can be filled during floods.

11.8.3 Information Dissemination

Whenever high outflow from the dam is likely, it is the duty of the in-charge of reservoir operation to alert the concerned authorities. The information must also be communicated to civil authorities, revenue authorities, police and general public for taking precautionary measures in respect of alerting and evacuating the people in the area likely to be affected.

To take full advantage of real-time operation, a good information dissemination system is necessary. In real-time operation, advance information can be given about the likely outflow and if the public is informed in time, the flood damage can be significantly reduced. The organizations responsible for issuing flood warning and flood fighting should be informed about the likely high releases from the dam as early as possible so that the required action is planned and activities set into operation with least possible delay. They should also be kept informed of the development of the high releases and any change in the present as well as anticipated future situation. The information is provided by the flood forecasting authorities in the form of *Bulletins*. These bulletins must be very clear and should include necessary details so that a realistic picture of the incoming danger is depicted. There should be arrangement to double-check the information supplied with clear-cut responsibilities in every office authorized to issue such bulletins. It is necessary to avoid dissemination of wrong information including even the inadvertent mistakes because hardly any time is left for review between dissemination of information and triggering a chain of follow-up activities.

11.8.4 Advantages of Real-time Operation

The advantages of real-time operation over the conventional methods are as follows:

- It is highly flexible compared to the conventional methods because the current state of the system and forecast of future inflows are into account while taking a decision.
- Real-time operation is the most realistic operation for reservoirs as operation decisions are frequently updated with the availability of new information.
- In emergencies, such as floods, it gives high lead-time to the authorities to take precautionary measures in respect of alerting and evacuating the area likely to be affected.

11.9 DEVELOPMENT OF OPERATING RULES FOR SABARMATI SYSTEM

For judicious regulation of water resources in the Sabarmati basin, operation policies are required for conservation and flood control purposes for the various hydraulic structures. A detailed description of the Sabarmati basin was given in Chapter 1. A line diagram of this system is given in Fig. 11.17. The following discussion illustrates the application of systems analysis techniques to develop operation procedures for the reservoirs in this system. It also brings out the difficulties that are faced while applying the techniques to a system where the data availability situation is somewhat unsatisfactory.

The aim of the conservation regulation of this system is to meet the demands in the best manner and avoid severe scarcity of water in the basin. In the flood season, the Dharoi reservoir is to be operated such that the total flow at Ahmedabad city does not exceed the safe carrying capacity of the river (14160 cumec). However, the safety of the dam is to be

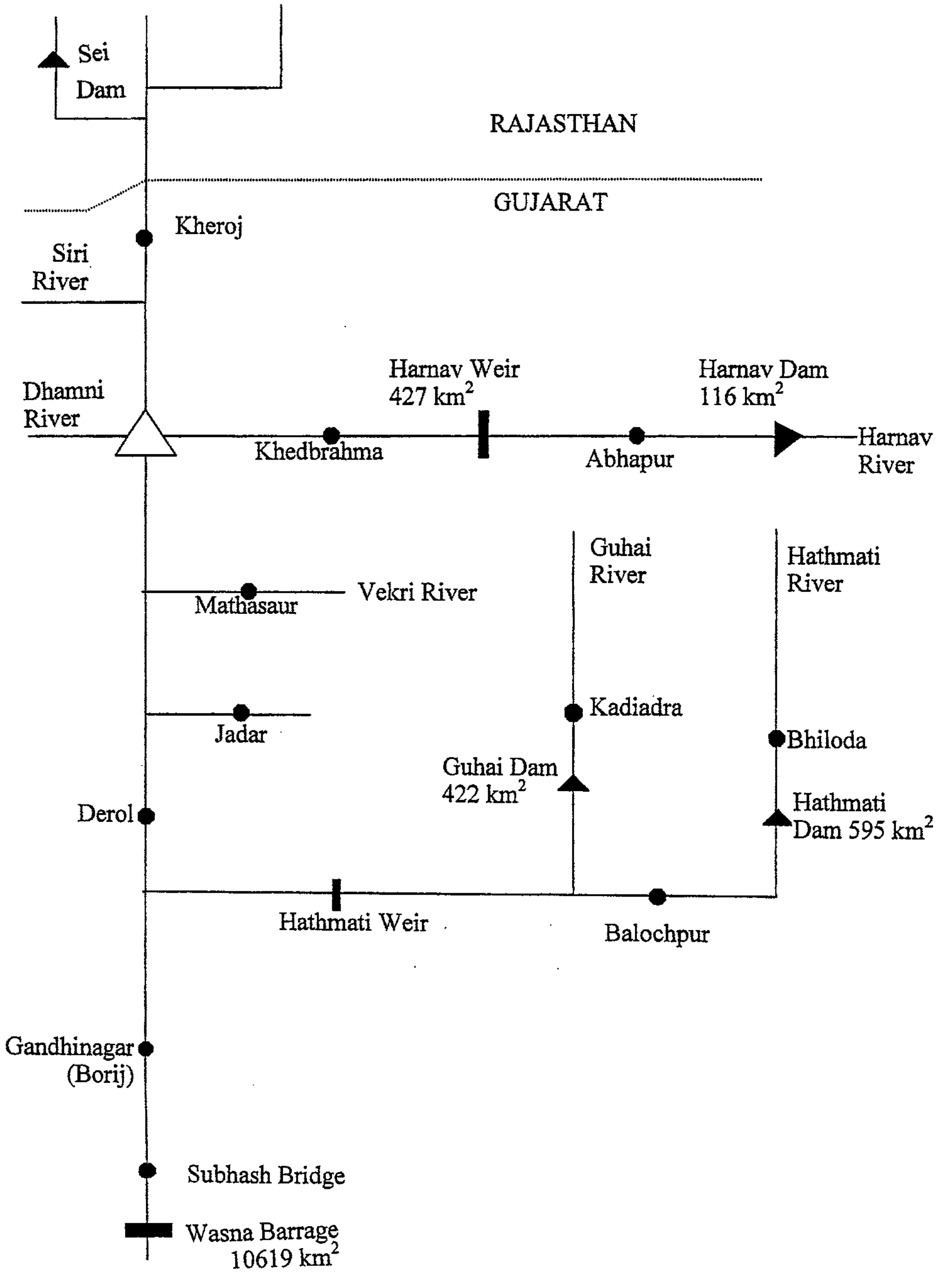

Fig. 11.17 Line Diagram of Sabarmati Basin up to Wasna Barrage (Ahmedabad).

given the top priority. The maximum discharge capacity of spillways at the Dharoi dam is 16992 cumec.

The simulation approach using historic observed flows was adopted to derive the regulation policy for various impoundments in the Sabarmati basin. Rule curves were derived for all the reservoirs serving various purposes. For flood regulation, the modelling of flow in the Sabarmati River at Ahmedabad was carried out and the release policy for the Dharoi dam was developed using the design flood hydrograph.

11.9.1 Solution Approach

A rule curve-based operation procedure was adopted for the conservation regulation of this system. Initial rule curves were derived for different reservoirs and these were refined using simulation.

A monthly time step was chosen to simulate the operation of the system. The operation of the system was simulated in accordance with the trial policy and calculates the release for different purposes, spill (if any) and the evaporation losses for each reservoir. During simulation, it takes into account the flow coming from upstream structures, the diversion of flow, and the release for meeting downstream demands. The output of the program includes a detailed working table and the time and volumetric reliability of each structure for the trial policy. It also calculates the number of critical failure months (release less than 75% of the target demand).

Since the flood control capacity of the structures in the Hathmati system was limited, the entire intermediate catchment (between the Dharoi dam and the Ahmedabad city) was assumed to be uncontrolled. For developing flood regulation procedure for the Sabarmati system, the modeling of flow at Ahmedabad was carried out. The flow at Ahmedabad was split into two components: the routed release from the Dharoi reservoir (which can be controlled) and the flow from the intermediate catchment (which cannot be controlled). Using the information about the releases from the Dharoi reservoir for the past few flood events, flow routing was carried out using the Muskingum-Cunge routing procedure (Ponce, 1989) and the various parameters of the method were calibrated. The flow from the intermediate catchment was modeled using the unit hydrograph approach.

This reservoir moderates the inflow in the space between the FRL at 189.59 m and the MWL at 193.60 m. The maximum observed flood at the Dharoi dam site was estimated to be 14158 cumec. In India, the design flood for major dams, such as Dharoi, is developed using the Probable Maximum Precipitation (PMP) approach. The design flood hydrograph for the Dharoi reservoir was used for developing the flood regulation procedure. The design storm is convoluted with the unit hydrograph for the catchment and the base flow is added to obtain the design flood hydrograph. The peak of the design flood hydrograph is 27180 cumec and its volume is 3095 Mm^3. It was felt that the flood regulation policy developed using the design flood hydrograph would be quite appropriate. A flood regulation simulation analysis was carried out for the Dharoi reservoir using different normal and emergency conditions under various scenarios of the downstream safe channel capacity.

11.9.2 Data Availability and Processing

Discharge is gauged in the basin at eight locations; the frequency of observation being hourly (during flood season) or once/twice/four-times a day. A sufficiently long inflow series at a structure was necessary for the simulation study. Usually, data for the period 1967 - 1993 were available. However, the virgin flows at some nodes were not available and the flows were estimated by adjusting the flow data at the nearest location in proportion to the catchment area, assuming that the catchment yield per unit area is the same. The target monthly demands, normal monthly evaporation depths and storage details for all the structures were collected from the concerned operating authorities.

A preliminary analysis of the Hathmati sub-basin flow data revealed that on the average, the annual flows in this basin were less than the annual target demands. Because of the deficit of water availability, any structure in this sub-system (Hathmati dam, Guhai dam and Hathmati weir) could not be used to supply water at the Ahmedabad/Wasna barrage. It was also not possible to divert water from any other structure to this sub-basin. Hence, the Hathmati sub-system was studied as an independent unit. The concurrent local flow data at the Guhai dam, the Hathmati dam and the Hathmati weir were processed for the period 1964 to 1993. For the Dharoi sub-system, consisting of the Harnav dam, the Dharoi dam and the Harnav weir, the average annual flows were more than the annual target demands. Hence, it was decided to meet some requirements of the Wasna barrage from the Dharoi reservoir. The common period of flow observation for the Dharoi sub-basin was from 1967 to 1993. Since for integrated operation, a common period of observation is necessary, the simulation analysis for the whole system was carried out for the period 1967 to 1993.

For a satisfactory flood regulation study, the modeling of flow in the Sabarmati River basin using short interval data (say, hourly) was necessary. Such data at various locations were available concurrently only after the year 1982. Those events were considered for which the hourly discharge data at various gauging stations were available concurrently with the hourly rainfall data at various self-recording raingauge stations. The hourly discharge data of Derol, Gandhinagar and Ahmedabad and the releases from the Dharoi reservoir were utilized for routing studies. Rainfall data of 11 raingauge stations were used to obtain the average hourly rainfall in the intermediate basin. Seven flood events, having consistent and concurrent short interval rainfall and discharge data, were used. These events were: July 23-26, 1982; August 11-20, 1983; August 4-7, 1984; August 11-15, 1984; July 19-22, 1988; August 4-6, 1988 and August 24-26, 1990.

11.9.3 Integrated Conservation Operation of the System

In the ranking of conservation demands, the domestic and industrial water supply demand is given the top priority while the irrigation demand is given the next priority. For conservation purposes, the system was to be operated such that: a) the water supply demand could be met to the extent possible; and b) the available irrigation water must be managed such that severe water shortages and subsequent crop failures can be minimized.

Derivation of Initial Rule Curves

More than 80% of the annual rainfall in India occurs in the four monsoon months from June to September. The reservoirs are filled during this period and the stored water is used throughout the water year. Using the power transformation approach, monthly inflows for 50%, 75% and 90% dependability, were estimated for each reservoir. Depending on the number, nature and priority of conservation demands, different rule curves were derived for each reservoir. For a reservoir serving for irrigation only, two rule curves, namely an upper rule curve and a lower rule curve, were derived. For a reservoir serving for water supply demand also, an additional rule curve for water supply was developed. The upper rule curve levels specify the maximum storage by month to which a reservoir should be filled if there is sufficient inflow. Lowering the upper rule curve below FRL provides storage for flood retention during the monsoon season, although it may affect the performance of the reservoir for conservation demands. Initially, the upper rule curve was set at FRL from June to September to ensure reservoir filling. In the months after the monsoon season, upper rule levels were calculated using 50% probable inflow, full target demands, evaporation losses, and uncontrolled spills from the reservoir using the forward computation:

$$\text{Storage}_{\text{end}} = \text{Storage}_{\text{begining}} + \text{Inflow} - \text{Demand} - \text{Evaporation} - \text{Spills} \qquad (11.42)$$

The evaporation losses were considered at normal monthly rate over the surface area of the reservoir corresponding to a particular elevation.

The middle/lower (whichever is applicable) rule curve is critical for irrigation demands and was derived for the situation when water is scarce and not all demands can be met throughout the water year. These were derived by assuming that the reservoir level will reach the dead storage level by the end of May. To compute them, 75% dependable inflow, full target demands, evaporation losses, and uncontrolled spills from the reservoir were considered and backward calculations were carried out starting from the end of May as:

$$\text{Storage}_{\text{begining}} = \text{Storage}_{\text{end}} - \text{Inflow} + \text{Demand} + \text{Evaporation} + \text{Spills} \qquad (11.43)$$

If the reservoir falls below this level in a particular month, the supply for irrigation must be suitably curtailed so that the reduced release can be made for a longer duration, thus avoiding severe crop failures. It is proposed that below this level, releases will be made to satisfy at the most 75% of the irrigation demands and full water supply demands (if any).

The lower rule curve levels (in reservoirs with water supply demand for domestic and industrial purposes) are critical for higher priority water supply demand. When the reservoir level falls below the lower rule level, the supply for irrigation must be completely curtailed and the release made only for the essential demands. This level was calculated using the approach mentioned for the middle rule level except that the 90% dependable inflow, only water supply demands, evaporation losses, and uncontrolled spills were considered. The trial rule curves were derived for all the reservoirs in a similar way.

Since the Dharoi reservoir serves irrigation and water supply demands, three rule curves were derived: upper rule curve, middle rule curve for full target demands and lower rule curve for water supply demands only. The Hathmati reservoir serves irrigation demands of its own command area and the higher priority irrigation demands of the command area of the Hathmati weir. Hence, three rule curves were derived to serve two demands of different priority. Since Hathmati dam is ungated and the reservoir is filled to FRL before any spill, there was no need to derive the upper rule curve and FRL at this reservoir corresponds to the upper rule level in all months. The middle rule curve was derived for the total irrigation demand while the lower rule curve was derived for the irrigation demands of the Hathmati weir. The other two reservoirs (Harnav and Guhai) have only irrigation demands in their own command area. So only two rule curves (upper and lower) were sufficient for these reservoirs.

Simulation of the System Operation

To derive the optimum rule curve reservoir levels (those giving the highest monthly time reliability with the least number of critical failure months), a detailed operation table was utilized. This table included the starting storage, virgin inflow, inflow from any upstream structure, demands from the structure, actual evaporation losses based on initial and final waterspread areas, release for different demands, spills and final elevation at each of the reservoirs for each month. For diversion structures, the simulation table included initial storage (in case of associated small storage structures), virgin inflow, inflow from upstream structures, demands at the structure, actual releases made, spills and the final storage. Initially, the upper rule curve levels were kept unchanged. Based on the observations from the simulation operation table, the initial lower rule curve levels were modified as long as the number of failure months (release < 100% of the target demand) could be reduced without increasing the number of critical failure months (release < 75% of the target demand). A number of simulation runs were made and the lower and the middle rule curve levels were tuned. Next, the trial upper rule curve levels were lowered in the four-monsoon months till there was no increase in the number of failure months.

Integrated operation of the Sabarmati system was simulated for the period 1967 to 1993. Based on the results from a number of trial runs, it was found that in the Hathmati sub-basin, the Hathmati dam could be operated to serve for a maximum of 70% of the target demand of the Hathmati weir. At the Hathmati weir, there exist two small storage structures, Limla dam and Karol dam with capacities of 10.28 and 7.5 M Cum, respectively, to store excess water. This water can serve the demands in the command area of the Hathmati weir. Before releasing the excess water in the downstream main Hathmati River, these dams are filled to their capacities. The operating authorities at the Dharoi reservoir retain storage reserve equivalent to one year water supply demand to cater for the eventuality of a monsoon failure in the following year. Thus, water is not released for irrigation from the Dharoi reservoir below the reservoir level 180.69 m. Trial simulation also demonstrated that only 50% of the Wasna barrage demands could be met from the Dharoi reservoir.

In the Dharoi sub-system, the monthly reliabilities of the Harnav and the Dharoi reservoirs for full assured supply were estimated to be 81.2% and 86.1%, respectively. For 75% assured irrigation supply, the monthly reliabilities for these structures were 93.8% and

90.1%, respectively. The monthly reliability of the Dharoi reservoir for water supply demand was 96.9%. The capacity of the Harnav reservoir (19.97 M Cum) is not adequate to support even the water supply demands at Dharoi. Furthermore, in periods of water scarcity at the Dharoi reservoir, water was also scarce in the Harnav dam. Hence, the Harnav dam may be operated to meet only its own demands. For the Harnav reservoir, in the span of 27 years, there were only 20 months when the release was less than 75% of the target demand. Most of these months occurred during the three acute drought years (1969, 1986 and 1987). Similarly, for the Dharoi reservoir, five years (1969, 1972, 1974, 1986 and 1987) were acute drought years in which 29 of the 32 critical failures occurred. Simulation showed that the present policy of the Dharoi operators of not supplying water for irrigation below 180.69 m (Dead Storage Level 175.87 m) could be relaxed in June and July as this improves the reliability of the reservoir with no increase in the number of months of water supply failure.

The simulation of the Hathmati sub-basin revealed that it is better to operate the Hathmati dam to meet 70% of the target Hathmati weir demand. The monthly reliability of the Guhai dam, the Hathmati dam and the Hathmati weir for meeting full target demand came out to be 61.4%, 54.6% and 59.6%, respectively. The years 1969, 1974, 1979, 1980, 1985 to 1990 and 1992 were severe drought years for this sub-basin and most of the failures occurred in these years. It was found that there is scarcity of water in this region and either target demands must be reduced and/or water conservation practices must be adopted.

As discussed earlier, the reservoir system operating policy meets the demands for as long a duration as possible. However, as soon as a shortage of water is anticipated at any time, the supply is curtailed so that the reduced supply could be maintained throughout the crop period. Simulation showed that 75% of the target demands could be met most of the time. The final rule curves for the Dharoi reservoir are shown in Fig. 11.18.

The upper rule curve implies that excess water above this level should be spilled from the reservoir to make some room for flood attenuation. This aspect of the system operation will be addressed in the next section. The middle rule curve implies a critical situation in the Dharoi reservoir for meeting full target demands in the remaining part of the water year. If the reservoir level drops below the middle rule curve, the authorities should initiate measures to avoid severe or sudden crop failure. The lower rule curve signifies the start of a critical water supply demand event (the highest priority demand). In this situation, releases for all other purposes should be curtailed to ensure that this highest priority demand is met to the fullest possible extent. When operating a reservoir using rule curves, it is advisable to periodically review and update the previous decision for the remaining duration of that month in light of new information, such as increased inflows, reduced demands, and so forth.

11.9.4 Flood Control Operation of the System

Reservoirs are frequently operated to control flooding at locations downstream of the dam. If the potential damage center is located far away downstream of the dam, a significant portion of the river flow at that location may be due to the contribution of the intermediate

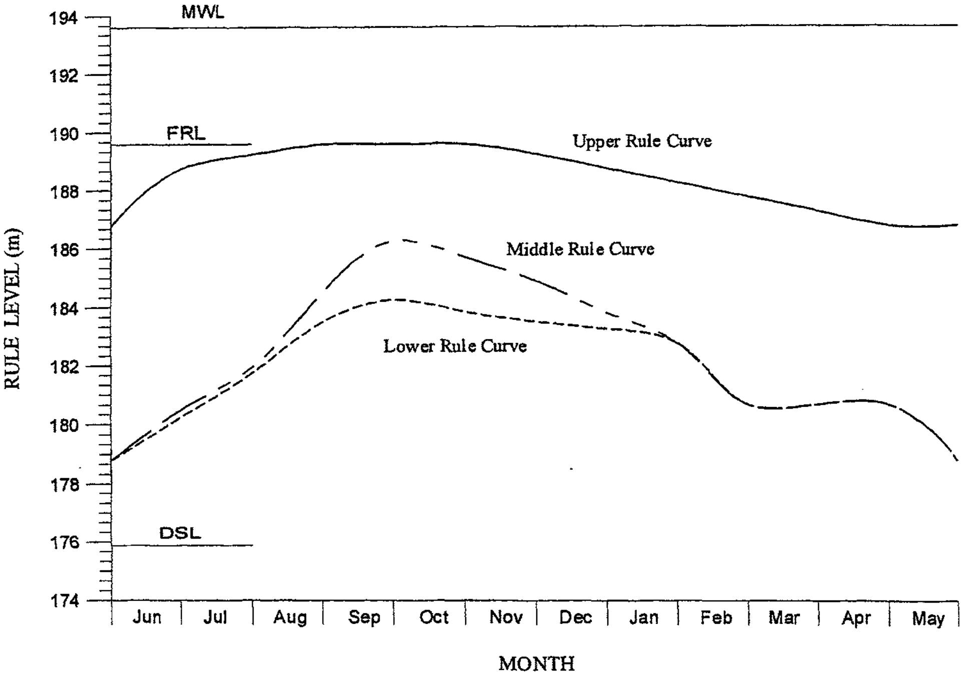

Fig. 11.18 Developed rule curves for the Dharoi reservoir.

catchment area. In such cases, it is necessary to determine the likely flow at the damage center due to the rainfall occurring in the intermediate catchment and the release from the upstream reservoir. Thus, modeling the flow at the damage center involves two steps: first, channel routing to analyze the effect of the release from the dam site; and second, the rainfall-runoff modeling of the intermediate catchment.

The Dharoi reservoir is the only major flood control storage in the Sabarmati basin. Since the flood retention capacity of the hydraulic structure in the Hathmati sub-basin was limited, the entire intermediate catchment (between the Dharoi dam and the Ahmedabad city) was assumed to be uncontrolled. In the Dharoi reservoir, space above FRL (189.59m) is exclusively for flood control. Therefore, it is proposed that this reservoir be operated for flood control only at elevations above 189.59 m. While deciding the release from the Dharoi reservoir, the operator must know the maximum safe release that can be made from the reservoir such that the flow at the Ahmedabad city does not exceed the desired limits.

Development of Flood Regulation Policy for Dharoi Reservoir

Simulation was adopted to derive the flood regulation policy. Various policies of the flood regulation were tried using different scenarios of the downstream safe channel capacity and different normal/emergency conditions in the reservoir. An exhaustive simulation analysis for the reservoir was carried out using the design flood hydrograph.

Through simulation, it was found that by keeping the emergency level at 191.00 m, the safe channel capacity at 11500 cumec and the spillway release capacity at 90% of the maximum (16990 cumec), it was possible to restrict the reservoir level below MWL (193.60 m). The analysis also suggests that as soon as a flood is anticipated, releases at a rate equal to the minimum of the safe channel capacity and the spillway release capacity at the current reservoir level, but more than the inflow rate must be made. This will create an extra empty space in the reservoir for flood attenuation.

The operation scenario of the Dharoi reservoir for flood regulation was classified in two categories: normal operation and emergency operation. Normal operation policy is applicable in any of the following conditions: i) reservoir level is below 191.00 m and the inflow rate less than 16992 cumec, ii) reservoir level is above 191.00 m and the inflow rate is less than 16992 cumec and it is decreasing. Under normal conditions, the maximum release from the reservoir would be equal to 50 to 80% of the safe carrying capacity of the downstream channel. This way, some additional empty space can be created in the reservoir for flood absorption. However, if the inflow starts decreasing, the release should be curtailed till the reservoir is filled to FRL. In case of (ii), the release should be such that the water level falls below the emergency level at the earliest.

The emergency operation is activated in any of the following conditions: i) reservoir level is above 191.00 m and either the inflow rate < 16992 cumec but increasing, or the inflow rate > 16992 cumec although decreasing; and ii) at any reservoir level, the inflow rate is more than 16992 cumec and it is increasing. Under emergency conditions, the release from the reservoir (may be more than the safe channel capacity) should be such that the reservoir level can be brought down to a safer level at the earliest, thereby avoiding overtopping of MWL. The minimum release rate under emergency conditions is the smaller of the safe channel capacity and the spillway release capacity.

With the developed flood regulation policy, it was found from the simulation analysis that by using the design flood hydrograph as inflow, the safe channel capacity of 9000 cumec and the maximum emergency release as 13900 cumec, the reservoir level could be maintained below MWL. Thus, as soon as a flood is anticipated in the reservoir, release at rates greater than the inflow rate but less than the safe channel capacity should be started. If the conditions in the reservoir are in the normal range, normal operation policy must be adopted. However, if the conditions in the reservoir reach the emergency state, emergency operation rules should be followed so that the safety of the structure can be ensured even if that entails exceeding channel capacity. The regulation of the design flood hydrograph through the Dharoi reservoir with the developed policy is presented in Fig. 11.19.

The likely hydrograph at Ahmedabad can be estimated by routing the Dharoi releases down the river up to Ahmedabad and adding the contribution of the intermediate catchment to it.

Routing of Reservoir Releases

Using the data about the releases from the Dharoi reservoir for the past flood events, flow

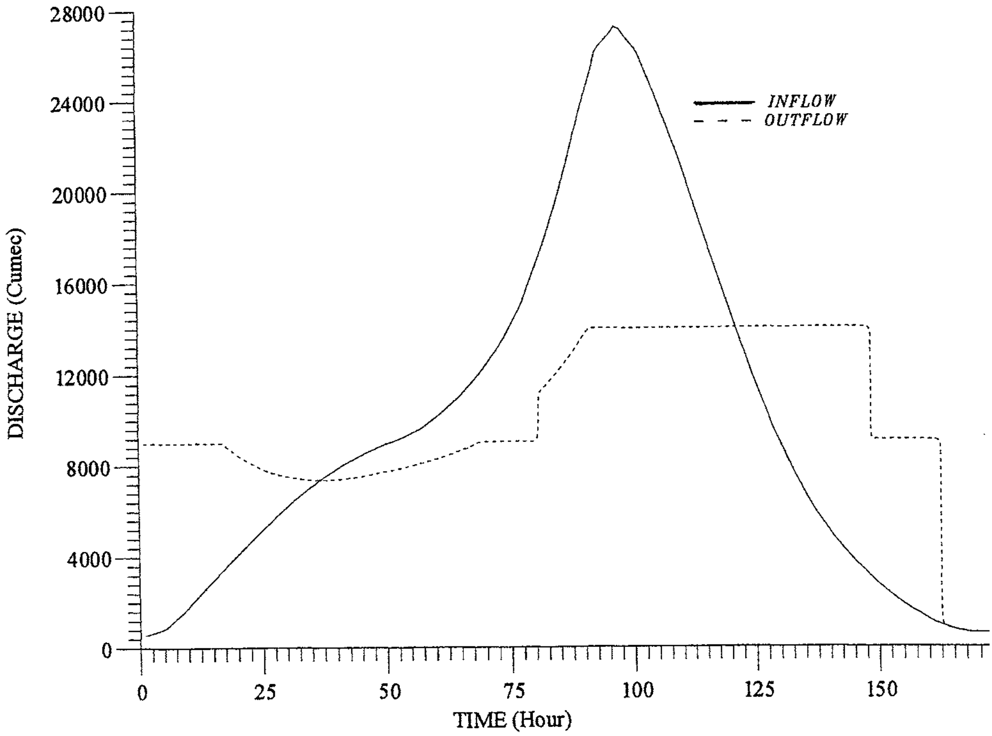

Fig. 11.19 Regulation of design flood hydrograph using the developed policy.

routing in the Sabarmati River was carried out from Dharoi/Derol up to Ahmedabad. The Muskingum-Cunge method (Ponce, 1989) of flood routing was used in this study. Two main reasons for adopting this technique were: a) the simplicity of the method, and b) this method makes use of topographic and cross-section information which was available for the present study.

Let the time be discretized in steps of Δt duration and the distance along the river in steps of Δx length. Following this method, if the inflow to the river reach at time $n\Delta t$ is denoted by I_n, the outflow Q_{n+1} at time $(n+1)\Delta t$ is given by

$$Q_{n+1} = C_0 I_{n+1} + C_1 I_n + C_2 Q_n \tag{11.44}$$

Define the Cell Reynolds number $D = q_0/(S_0\, c\, \Delta x)$, and the Courant number $C = c\, \Delta t/\Delta x$, where S_0 is the channel bottom slope, q_0 is the reference flow per unit channel width and c is the wave celerity ($c = \beta V$, β is the rating exponent and V is the average velocity corresponding to peak flow). The coefficients in the eq. (11.44) are:

$$C_0 = \frac{-1+C+D}{1+C+D} \qquad C_1 = \frac{1+C-D}{1+C+D} \qquad C_2 = \frac{1-C+D}{1+C+D} \tag{11.45}$$

The methodology would be applicable, subject to the satisfaction of the criteria: $T_rS_oV_o/D_o \geq 85$, where T_r is the time to peak of the inflow hydrograph, V_o is the average flow velocity, and D_o is the average flow depth. The value of Δt is chosen such that $\Delta t \leq \Delta x/c$ and $\Delta t \leq T_r/5$. The value of Δx should be chosen such that $q_o\Delta t/(0.25\ S_o) \geq \Delta x \geq q_o/(S_oc)$. To ensure numerical stability, the values of Δx and Δt should satisfy the condition $C+D \geq 1$.

During a flood, the time available to the dam operator for deciding various parameters of the Muskingum-Cunge routing procedure is short. For this reason, estimates of different parameters corresponding to various peak discharges were developed. The number of reaches to be considered for routing was found to vary inversely with the discharge in the river. From Dharoi to Ahmedabad, the number of required reaches was found to be in the range from 13 to 35. The top width of the river, the flow area and the rating exponent were also calibrated for different peak discharge values. The time step (Δt) equal to half-hour was found suitable for satisfying various conditions as mentioned above. Using a half-hourly time step, space step (Δx) values are calculated. Using these values, one can estimate the routed flow at Ahmedabad corresponding to any trial release from the Dharoi reservoir.

Flood routing was carried out for those events for which the concurrent flow data were available at various gauging stations. For routing between Derol and Ahmedabad, the observed hourly discharge data at Derol, Gandhinagar and Ahmedabad were used. The Hathmati River is the only major tributary that joins the Sabarmati River downstream of, but very near to, the Derol gauging site. Thus, a large volume of lateral inflow enters the river Sabarmati between Derol and Ahmedabad. There is no gauging site on the Hathmati River near its confluence with Sabarmati. The lateral inflow hydrograph from the Hathmati River was estimated by routing the flow at Derol up to Gandhinagar and then subtracting the routed flow from the observed flow at Gandhinagar.

For the flood events for which the release data from the Dharoi reservoir were available, the flow from the intermediate catchment was calculated by routing the release from Dharoi dam up to Ahmedabad and then subtracting the routed flow from the observed flow at Ahmedabad. For the events when the reservoir release was nil, the entire flow at Ahmedabad was assumed to be the contribution from the intermediate catchment. These computed flows were used for rainfall-runoff modeling of the intermediate catchment.

Modeling of the Intermediate Catchment

The catchment area between the Dharoi dam and Ahmedabad city, including the Hathmati sub-basin, is 5079 sq. km. To model the contribution from a catchment of this size, it would be appropriate to carry a detailed modeling by subdividing it into sub-catchments but the required short-term rainfall and discharge data of sufficient duration for each sub-catchment were not available. In view of limited data, it was decided to develop a unit hydrograph for the whole intermediate catchment. Release data from the Dharoi reservoir were available for seven events and these were used for modeling the intermediate catchment. The average hourly rainfall for the intermediate catchment was estimated using the Thiessen Polygon method. Since the entire intermediate catchment was modeled as a single unit, the total lateral flow hydrograph at Ahmedabad, computed during the routing analysis, was used as the observed flow.

The unit hydrograph was developed by using Snyder's synthetic unit hydrograph method. The average hourly rainfall over the intermediate catchment was estimated as discussed above. The volumes of the observed lateral flow and the observed rainfall were computed and the runoff coefficient corresponding to different rainfall depths was calibrated. The runoff coefficient for the intermediate catchment was found to be in the range from 0.06 to 0.15. A synthetic unit hydrograph was developed for the intermediate basin and convoluted with the effective hourly rainfall. The observed and computed hydrographs were compared and the unit hydrograph was modified until a satisfactory response was achieved.

Software to Calculate Safe Release

A computer program was developed to calculate the flow hydrograph at Ahmedabad city resulting from the release from the dam and the rainfall in the intermediate catchment. The program uses the Muskingum-Cunge routing procedure and the unit hydrograph for this purpose. This program can assist the operator at the dam site in deciding the safe release from the reservoir at any time. The input to the program includes the past half-hourly reservoir releases, present trial release and the average rainfall in the intermediate catchment during all the past hours of the current flood event. The past reservoir releases and past average rainfall depths are stored in relevant files. The various calibrated parameters (space step, number of reaches, top channel width, flow area, wave celerity, and runoff coefficient) have been specified in the program. The output from the program is the release from the current time period.

The data availability was the main constraint in the application of improved techniques to this real-life problem. In addition, the accuracy and authenticity of the data collection system created additional problems in successfully modeling the Sabarmati River basin system.

11.10 CLOSURE

Due to the increase in population and urbanization, the conservation demands as well as damage potential of floods are increasing day-by-day but it is not easy to create new reservoirs because of social and environmental ramifications. Therefore, it is essential to operate the existing reservoirs as efficiently as possible. Real-time operation is an efficient way of operating a reservoir system in which the control decisions are made on the basis of prevailing conditions of the system and the forecast about the likely inflow in the reservoir. For real-time operation of a reservoir, automatic telemetry system is essential for direct transmission of data at a regular interval to the forecasting station from where forecasts are issued. This hydrological forecasting is used as input in the operation model to find the optimized value of the release from the reservoir.

Many smaller reservoirs are operated using simple 'common-sense' rules such as keeping reservoirs full for conservation purposes or empty to control floods. Rules for major projects are derived using systems analysis techniques. However, in general simulation appears to be the technique that is most common in deriving procedures that are used in practical situations. Despite significant developments in the reservoir operation area, many reservoirs are still operated using procedures that have not been developed 'scientifically'.

A lot of work on real-time operation is going on and efforts are on to develop models which can capture the details of the system in as much detail as possible. Computation time is an important aspect in real-time operation as the 'lead time' of a forecast is affected by it. Due to the increasing availability of high capability and low cost computers with telemetry networks and remote control features, the hardware required for this purpose is easily available. A number of models using different optimization techniques, such as linear programming, dynamic programming and goal programming have been developed and applied to various real life problems in the form of case studies. A considerable amount of software is also available. A number of decision support systems have also been developed which can assist the operator in making better decisions.

11.11 REFERENCES

Bhasker, N.R., and Whitlatch, E.E. (1987). Comparision of reservoir linear operation rules using linear and dynamic programming. Water Resources Bulletin, 23(6), 1027-1036.

Biswas,A.K.(Ed.) (1976). Systems Approach to Water Management, McGraw Hill Book Company, New York.

Biswas, A.K. (1991). Water for sustainable development in the 21st century: A global perspective. Water International, 6(4), 219-224.

Butcher, W. (1971). Stochastic dynamic programming for optimum reservoir operation", Water Resources Bulletin, 7(1), 115-123.

Datta, B. and Burges, S.J. (1984). Short-term single, multiple purpose reservoir operation. Water Resources Res., 20(9), pp. 1167-1176.

Eastman, J., and ReVelle, C. (1973). Linear decision rule in reservoir management and design, 3. Direct capacity determination and intraseasonal constraints. Water Resources Research, 9(1), 29-43.

Feldman, A.D. (1981). HEC models for water resources system simulation: Theory and experience. Advances in Hydroscience (V.T. Chow, ed.), Vol. 12, Academic Press, New York.

Fontane, D., Labadie, J.W., and Loftis, B. (1981). Optimal control of reservoir discharge quality through selective withdrawal. Water Resources Research, 17(6), 1594-1604.

Goulter, I.C., and Tai, F. K. (1985). Practical implications in the use of stochastic dynamic programming for reservoir operation. Water Resources Bulletin, 21(1), 65-74.

Gundelach, J., and ReVelle, C. (1975). Linear decision rules in reservoir management and design 5:A general algorithm. Water Resources Research, 11(2), 204-207.

Hall, W.A., and Dracup, J.A. (1970). Water Resources System Engineering, McGraw-Hill Book Company, New York.

HEC (1976). Flood control by reservoirs. Hydrologic engineering methods for water resources development, Vol. 7, Hydrologic Engineering Center, Davis, USA.

HEC (1982). HEC - 5 Simulation of Flood Control and Conservation System, Program Users Manual, Hydrologic Engineering Center, Davis, USA.

Jain, S.K., and M.K. Goel (1997). Software for Reservoir Analysis, Report No. UM-1/96-97. National Institute of Hydrology, Roorkee.

Jain, S.K., and M.K. Goel (1999). Flood Control Regulation of a Multi-reservoir System, National Institute of Hydrology, Roorkee.

Jain, S.K., Goel, M.K., and Agarwal, P.K. (1998). Reservoir operation studies for Sabarmati system, India. Journal of Water Resources Planning and Management, ASCE, 124(1), 31-38.

Jain, S.K., Das, A., and Srivastava, D.K. (1999). Application of ANN for reservoir inflow prediction and operation. Journal of Water Resources Planning and Management, ASCE, 125(5), 263-271.

Jairaj, P.G., and Vedula, S. (2000). Multireservoir system optimization using fuzzy mathematical programming. Water Resources Management, 14, 457-472.

James, L.D. and Lee, R.R. (1971). Economics of Water Resources Planning, McGraw-Hill Book Co., New York.

Joeres, E.F., G.J. Seus, and H.M. Engelmann (1981). The linear decision rules (LDR) reservoir problem with correlated inflows 1: Model development. Water Resources Research, 17(1), 18-24.

Johnson, S. A., Stedinger, J. R., and Staschus, K. (1991). Heuristic operating policies for reservoir system simulation. Water Resources Research, 27(6), 673- 685.

Kite, G.W. (1977). Frequency and Risk Analysis in Hydrology, Water Resources Publications, USA.

Karamouz, M. and M.H. Houck (1987). Comparison of stochastic and deterministic dynamic programming for reservoir operating rule generation. Water Resources Bulletin, 23(1), 1-9.

Kuiper, E. (1965). Water Resources Development, Butterworths and Company Ltd., London.

Lettenmaier, D.P., and Wood, E.F. (1993). Hydrological forecasting, in Handbook of Hydrology. Edited by D.R. Maidment. McGraw-Hill Book Company, New York.

Loucks, D.P., and P. Dorfman (1975). An evaluation of some linear decision rules in chance constrained models for reservoir planning and operation. Water Resources Research, 11(6), 777-782.

Loucks, D.P. and O.T. Sigvaldason (1980). Operations research in multiple reservoir operation. In Operations Research in Agriculture and Water Resources, Edited by D. Yaron and C. Tapiero, North Holland Publishing Company.

Loucks, D. P., and Sigvaldason, O. T. (1982). Multiple-reservoir operation in North America, in The operation of multiple reservoir systems, Z. Kaczmarek and J. Kindler, editors. International Institute for Applied Systems Analysis, Laxenburg, Austria.

Loucks, D.P., Stedinger, J.R. and Haith, D.A. (1981). Water Resources Systems Planning and Analysis, Prentice Hall Inc., New Jersey.

Loucks, D.P., Salewicz, K.A. and Taylor, M.R. (1989). IRIS: An Interactive River System Simulation Model, General Introduction and Description. International Institute for Applied Systems Analysis, Laxenburg, Austria.

Lund, J. R., and Ferreira, I. (1996). Operating rule optimization for the Missouri River reservoir system. Journal of Water Resources Planning and Management, ASCE, 122(4), 287–295.

Lund, J.R. and Guzman, J. (1999). Derived operating rules for reservoirs in series or in parallel. Journal of Water Resources Planning and Management, ASCE, 125(3), 143-153.

Maass, A., Hufschmidt, M.M., Dorfman, R., Thomas, H.A., Marglin, S.A., and Fair, G.M. (1962). Design of Water Resource Systems, Harvard University Press, USA.

Martin, Q. W. (1982). Multivariate simulation and optimization model (SIM-V), Program documentation and user's manual. UM-38, Texas Department of Water Resources, Austin, USA.

McMahon, T.A., and Mein, R.G. (1986). River and Reservoir Yield. Water Resources Publications, Colorado, USA.

Marien, J. L., Damazio, J. M., and Costa, F. S. (1994). Building flood control rule curves for multipurpose multireservoir systems using controllability conditions. Water Resources Research, 30(4), 1135– 1144.

Nagy, I.V., Asante-Duah,K., and Zsuffa, I. (2002). Hydrological dimensioning and operation of reservoirs. Water Science and Technology Library, Kluwer Academic Publishers, Dordrecht.

Oliveira, R., and Loucks, D. P. (1997). Operating rules for multireservoir systems. Water Resources Research, 33(4), 839– 852.

ReVelle, C., Joers, E. and Kirby, W. (1969). The linear decision rule in reservoir management and design, 1. Development of the stochastic model. Water Resources Research, 5(4), 767-777.

ReVelle, C., and W. Kirby (1970). Linear decision rule in reservoir management and design, 2. Performance optimization. Water Resources Research, 6(4), 1033-1044, 1970.

ReVelle, C., and J. Gundelach (1975). Linear decision rule in reservoir management and design, 4. A rule that minimizes output variance. Water Resources Research, 11(2), 197-203.

ReVelle, C. (1999). Optimizing reservoir resources, including a new model for reservoir reliability. John Wiley and Sons, New York.

Sheer, D. (1986). Dan Sheer's reservoir operating guidelines. School of Civil and Environmental Engineering, Cornell University, Ithaca, N.J.

Sigvaldason, O.T. (1976). A simulation model for operating a multi-purpose multi-reservoir system. Water Resources Research, 12(2), 263-278.

Simonovic, S.P. (1992). Reservoir system analysis: Closing gap between theory and practice. Journal of Water Resources Planning and Management, ASCE, Vol. 118(3), 262-280.

Stedinger, J. R., Sule, B. F., and Pei, D. (1983). Multiple reservoir system screening models. Water Resources Research, 19(6), 1383– 1393.

Unny, T.E., and McBean, E.A. (ed) (1982). Decision Making for Hydrosystems: Forecasting and Operation. Water Resources Publications, Littletown, USA.

USACE (1985). Engineering manual: Engineering and design, Hydropower. (1985). EM 1110-2-1701, U.S. Army Corps of Engineers, Washington, D.C.

USACE (1987). SSARR Users Manual: Streamflow Synthesis and Reservoir Regulation, US Army Corps of Engineers, Portland, Oregon, USA.

Windsor, J.S. (1975). A programming model for the design of multireservoir flood control systems. Water Resources Research, 10(1), 30-35.

Wurbs, R.A. (1993). Reservoir system simulation and optimization models. J. Water Resources Planning and Management, ASCE, Vol. 119(4), 455-472.

Wurbs, R.A., Dunn, D.D., and Walls, W.B. (1993). Water Rights Analysis Program (TAMU WRAP), Model Description and Users Manual, Tech. Rep. 146, Texas Water Resources Institute, College Station, Texas, USA.

Wurbs, R.A. (1996). Modeling and Analysis of Reservoir System Operations, Prentice Hall, New York.

Yakowitz, S. (1982). Dynamic programming applications in water resources. Water Resources Research, 18(4), 673-696.

Yeh, W.W-G. (1985), Reservoir management and optimization models: a state-of-the-art review. Water Resources Research, 21(12), 1797-1818.

Yeh, W.W-G., and Becker, L. (1982). Multiobjective analysis of multireservoir operation. Water Resources Research, 18(5), 1326-1336.

Young, G.K. (1967).Finding reservoir operating rules. Journal of the Hydraulics Division, ASCE, 93(HY6), 297-321.

Zadeh, L.A. (1965). Fuzzy sets. Information and Control, 8, 338-353.