11. River Basin Planning Models

11 River Basin Planning Models

Multipurpose river basin management typically involves the identification and use of both structural and non-structural measures designed to increase the reliability of municipal, industrial and agriculture water supplies when and where demanded, to protect against floods, to improve water quality, to provide for commercial navigation and recreation, and to produce hydropower, as appropriate for the particular river basin. Structural measures may include diversion canals, reservoirs, hydropower plants, levees, flood proofing, irrigation delivery and drainage systems, navigation locks, recreational facilities, groundwater wells, and water and wastewater treatment plants along with their distribution and collection systems. Non-structural measures may include land use controls and zoning, flood warning and evacuation measures, and economic incentives that affect human behaviour with regard to water and watershed use. Planning the development and management of water resources systems involves identifying just what, when and where structural or non-structural measures are needed, the extent to which they are needed, and their combined economic, environmental, ecological and social impacts. This chapter introduces some modelling approaches for doing this. The focus is on water quantity management. The following chapter reviews some measures and models for water quality management.

1. Introduction

This chapter introduces some types of models commonly used to assist those responsible for planning and managing various components of river systems. These components include the watersheds that drain into the surface water bodies and underlying aquifers of river systems. They include the streams, rivers, lakes, reservoirs and wetlands that can exist in river basins and that are affected by water management policies and practices. First, each of these components will be examined and modelled separately. The management of any single component, however, can affect the performance of other components in a river basin system. Hence, for the overall management of river basin systems, a systems view is needed. Typically, this systems view requires the modelling of multiple components. These multi-component models can then be used to analyse alternative designs and management strategies for integrated multi-component systems.

River basin planning is a prerequisite for integrated water resources management (IWRM). IWRM requires the integration of the natural system components (surface water–groundwater, quantity–quality, land- and water management, etc.) and the upstream and downstream water-related demands or interests. Water resources planning is increasingly done on a river basin scale. The European Water Framework Directive, for example, imposes the development of basin plans in Europe, forcing riparian countries to work together on the development and management of their river basins.

The discussion in this chapter is limited to water quantity management. Clearly the regimes of flows, velocities, volumes and other properties of water quantity will affect the quality of that water as well. Moreover, from the perspective of IWRM, quantity and quality aspects should be considered jointly. However, unless water quantity management strategies are based on requirements for water quality, such as for the dilution of pollutants, water quality measures do not normally affect water quantity. For this reason it is common to separate analytical approaches of water quantity management from those of water quality management. However, when attempting to predict the impacts of any management policy on both water quantity and quality, both quantity and quality models are needed.

This chapter begins with an introduction on the scales of river basin processes and an overview of modelling approaches. Section 2 describes the modelling of the natural resources system (see also Figure 1.19 in Chapter 1). This includes the estimation of unregulated surface and groundwater runoff from watersheds, as well as the prediction of streamflows at various sites of interest throughout a basin. This section also includes several methods for estimating reservoir storage requirements for water supplies and flood storage. Reservoirs can provide for water supply, flood control, recreation and hydroelectric power generation. These 'use-functions' and their modelling are described in Section 3 on socio-economic functions. The natural resource system and the socio-economic functions are then, in Section 4, combined into a multiple-purpose multi-objective planning model for a river basin. The chapter concludes with an introduction to some dynamic models for assisting in the scheduling and time sequencing of multiple projects within a river basin.

1.1. Scales of River Basin Processes

One of the challenges in constructing integrated models of multiple river basin processes is the wide range of spatial and temporal scales that characterize these processes. Precipitation, which is the source of virtually all freshwater in the hydrological cycle, is typically highly variable and uneven in its distribution over time and space. Similarly, the rates of evaporation and transpiration vary considerably according to climatic and land-cover conditions. The relative magnitudes of the fluxes associated with individual components of the hydrological cycle, such as evapotranspiration, may differ significantly even at small spatial scales such as on a commercial shopping mall, an agricultural field and a woodland. Surface-flow conditions that are of interest to those managing water supplies, navigation, hydropower production, recreation and water quality can be modelled using mass balances in one-dimensional models with time steps of up to a season in length. Flood-flow conditions on the other hand require higher dimension models and much smaller time steps. Groundwater flows can be modelled with much longer time steps than surface water flows but, if both surface and groundwater bodies interact, some way has to be developed to make these different time scales compatible.

Figure 11.1 attempts to identify the range of spatial and temporal scales usually considered by those who model these various components and processes. Much depends, however, on just what impacts are to be identified and on which variables are considered unknown and which are assumed known. For example, it is not uncommon to model water supply systems that include reservoirs and diversions to water consumers on a monthly or even seasonal time-step scale. However, if it is important to capture the within-month changes in reservoir storage for recreation or hydropower, smaller time steps may be necessary during those months or seasons where the variation in storage and flows is important. If water quality impacts are being estimated, time steps of a day or less may be needed, depending on the detail or precision desired. Some watershed processes, such as erosion and evapotranspiration, are best modelled on an hourly or minute-by-minute basis if sufficient data are available to justify those short time steps, whereas other watershed processes, such as changing ownership and land cover, may be modelled using ten-year or longer time steps. Much depends on the variability of both the meteorological and hydrological inputs (air temperatures, radiation, precipitation and streamflows) and the output targets (demands). If there is not much variation during a particular season, it may not be necessary to define smaller time periods within that season.

Figure 11.1 shows the ranges of spatial and temporal scales usually considered when modelling the processes taking place in the three major river basin components – watersheds, surface water bodies and groundwater aquifers. There are exceptions. The boundaries of each zone shown in Figure 11.1 are not as clearly defined as the figure would suggest.

To provide some perspective on spatial extent of some rivers and their watersheds, Table 11.1 lists a few of the

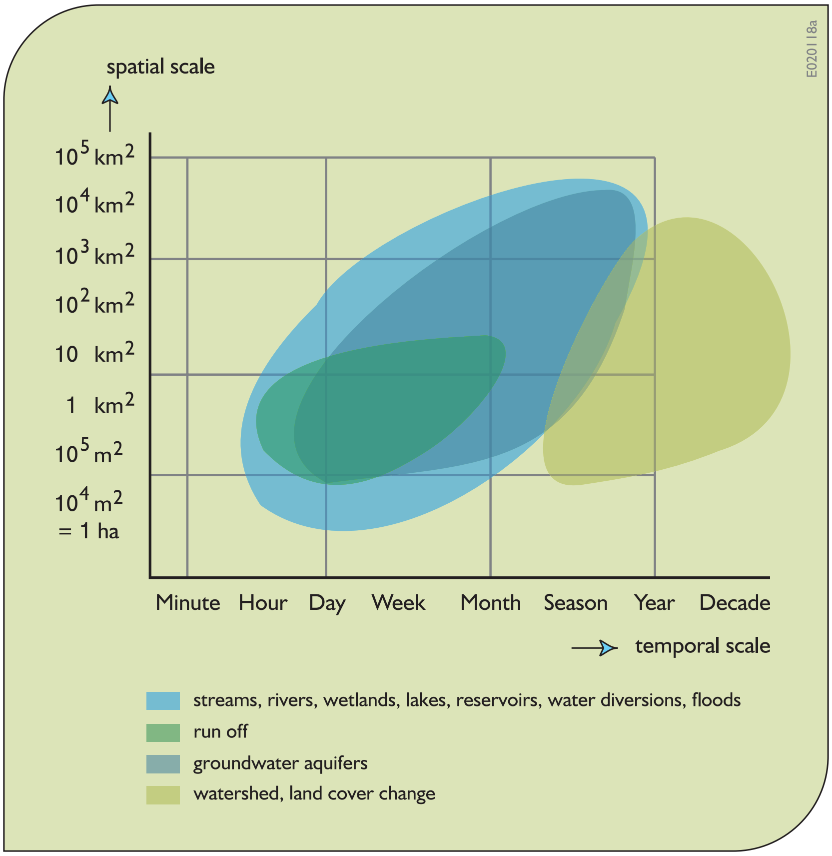

Figure 11.1. Common spatial and temporal scales of models of various river basin processes.

world's major rivers with their watershed areas and lengths.

1.2. Model Time Periods

When analysing and evaluating various water management plans designed to distribute the natural unregulated flows over time and space, it is usually sufficient to consider average conditions within discrete time periods. Commonly in river basin modelling, weekly, monthly or seasonal flows are used as opposed to daily flows. The shortest time-period duration usually considered in models developed for planning analyses is one that is no less than the time water takes to flow from the upper to the lower end of the river basin. In this case stream and river flows can be defined by simple mass-balance or continuity equations. For shorter duration time periods, some kind of flow routing may be required.

The actual length of each within-year period defined in a model may vary from period to period. The appropriate number and durations of modelled time periods will depend, in part, on the variation of the supply of natural flows and the variation of demands for water flows or storage volumes.

Another important factor to consider in making a decision as to the number and duration of periods to include in any model is the purpose for which the model is to be used. Some analyses are concerned only with identifying the desirable designs and operating policies of various engineering projects for managing water resources at some fixed time (a year, for instance) in the future. Multiple years of hydrological records are simulated to obtain an estimate of just how well a system might perform, at least in a statistical sense, in that future time period. The within-year period durations can have an impact on those performance indicator values, as well as on the estimate of over-year as well as within-year storage requirements that may be needed. These static analyses are not concerned with investment project scheduling or sequencing. Dynamic planning models include changing economic, environmental and other objectives and design and operating parameters over time. As a result, dynamic

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river	continent	area 10^3 km^2	length km
Amazon	S. America	6915.0	6280
Congo	Africa	3680.0	4370
La Plata	S. America	3100.0	4700
Ob Asia	Asia	2990.0	3650
Mississippi	N. America	2980.0	6420
Nile	Africa	2870.0	6670
Yenisei	Asia	2580.0	3490
Lena	Asia	2490.0	4400
Niger	Africa	2090.0	4160
Amur	Asia	1855.0	2820
Yangtze	Asia	1800.0	5520
Mackenzie	N. America	1790.0	5472
Gangas	Asia	1730.0	3000
Volga	Europe	1380.0	3350
Zambezi	Africa	1330.0	2660
St.Lawrence	N. America	1030.0	3060
Orinoco	S. America	1000.0	2740
Indus	Asia	960.0	3180
Yukon	N. America	850.0	3000
Danube	Europe	817.0	2860
Mekong	Asia	810.0	4500
Hwang Ho	Asia	745.0	4670
Columbia	N. America	668.0	1950
Kolyma	Asia	647.0	2130
Sao-Francisco	S. America	623.0	2800
Dnepr	Europe	504.0	2200
Chutsyan	Asia	437.0	2130
Indigirka	Asia	360.0	1726
N.Dvina	Europe	357.0	744
Pechora	Europe	322.0	1810
Godavari	Asia	314.0	1500
Neva	Europe	281.0	74
Magdalena	S. America	260.0	1530
Krishna	Asia	256.0	1290
Fraser	N. America	233.0	1370
Ogowe	Africa	210.0	850
Essequibo	S. America	155.0	970
Sanaga	Africa	135.0	860
Narmada	Asia	102.0	1300
Rhine	Europe	99.0	810
Ebro	Europe	86.8	930
Atrato	S. America	32.2	644
San Juan	S. America	21.5	430

Table 11.1. Some of the world's major rivers and their spatial characteristics.

models generally span many more years than do static models, but they may have fewer within-year periods.

1.3. Modelling Approaches For River Basin Management

Chapter 3 presented a general overview of modelling methods. The distinction made in that chapter between simulation and optimization approaches applies also to river basin planning. The complexity involved in river basin planning requires both approaches. In the design phase of planning studies, optimization methods are often useful for making a first selection among the countless options for capacities and management measures. However, applying optimization requires simplifications, in particular with respect to spatial and temporal detail. For this reason, simulation models should be applied to check and refine the infrastructural designs and operating policies as well as to develop detailed management options. Reference is made to Chapter 3 (Section 3.4) for a more detailed discussion of simulation and optimization models.

Both optimization and simulation modelling approaches require a description of the natural resource system and related infrastructure, and a description of the socio-economic functions in the river basin. These model components will be introduced in the next two sections. In Section 4 these two components are combined for more comprehensive river basin analyses.

2. Modelling the Natural Resource System and Related Infrastructure

The natural resource system (NRS) comprises all natural physical, chemical and biological aspects and related artificial elements of the water system. This chapter focuses on the quantitative aspects; water quality and ecology will be described in more detail in Chapter 12. The hydrological surface water and groundwater system will be described, as well as the function of reservoirs to augment the supply in periods of low rainfall and river flows and to contain flood flows in periods of high river flows.

2.1. Watershed Hydrological Models

Hydrological modelling is used to predict runoff from land areas, infiltration into soils and percolation into aquifers. Rainfall–runoff models are often used when streamflow gauge data are not available or not reliable, or cannot be made representative of natural flow conditions (that is, what the flows in a stream or river would be without upstream diversions or reservoirs that alter the flows downstream). They are also used to provide estimates of the impact that changing land uses and land covers have on the temporal and spatial distribution of runoff.

2.1.1. Classification of Hydrological Models

Hydrological models are classified as either *theoretical* or *empirical* models. A theoretical model is based on physical principles. If all the governing physical processes are described by mathematical functions, a model containing those functions is a physically based model. However, most existing hydrological models simplify the physics and often include empirical components. For example, the conservation of momentum equation or Manning's equation for predicting surface flow include empirical hydraulic resistance terms. Darcy's equation, used to predict subsurface flows, requires an empirical hydraulic conductivity parameter value. Thus they are considered at least partially, if not fully, empirical. Purely empirical or statistical models (Chapter 6) omit the physics and are in reality representations of the observed data.

Depending on the character of the results obtained, hydrological models can also be classified as *deterministic* or *stochastic*. If one or more of the variables in a mathematical model are regarded as random variables whose values can change unpredictably over time, then the model is stochastic. If all the variables are considered to be free from random variation, the model is deterministic. Of course some 'deterministic models' may include stochastic processes that capture some of the spatial and temporal variability of some of the sub-processes, such as infiltration. Most hydrological models are deterministic in spite of a host of random processes taking place in the watersheds to which they are applied. The rather simple modelling approach outlined below for estimating the relative surface and groundwater runoff to surface water bodies is an example of a deterministic model.

Hydrological models can also be classified as *event-based models* or as *continuous-time models*. An event-based model simulates a single runoff event, such as a single storm, usually occurring over a period of time ranging from about an hour to several days. The initial conditions in the watershed for each event must be assumed or determined by other means and supplied as input data. The accuracy of the model output may depend on the reliability of these initial conditions. A continuous-time watershed model includes a sequence of time periods and for each period determines the state of the watershed, whether or not any events take place that will produce surface runoff. The model keeps a continuous account of the watershed surface and groundwater conditions. The effect of any assumed initial conditions decreases rapidly as time advances. Most continuous watershed models include three water balances – one for surface water, another for unsaturated zone moisture content, and a third for groundwater. An event model may omit one or both of the subsurface components and also evapotranspiration when those losses are small in relation to the surface runoff.

Finally, models that have been developed to simulate hydrological processes can be classified as either *lumped* or *distributed* or a mix of both. Lumped models assume homogenous or average conditions over all or portions of a watershed. Lumped models are not sensitive to the actual locations of the varying features in the watershed. Distributed models take into account the locations of various watershed conditions such as land covers, soil types and topography when estimating the total runoff. Both types of models are useful. Distributed models require more detailed data, but are needed if, for example, one wishes to evaluate the impacts of riparian (tree-belt) buffers along streambanks, or the effects of varying topographic features within the watershed. Some models are mixes of the two types of models, in other words, quasi- or semi-distributed models made up of multiple connected lumped models representing different parts of watersheds.

2.1.2. Hydrological Processes: Surface Water

The manner and detail in which various hydrological processes are modelled varies among different models and their computer programs. Models that are developed to better understand the physical and other natural processes taking place on and under the surface of a watershed are usually much more detailed than models used for sizing drainage ditches or culverts or for evaluating alternative watershed land use management policies or practices. Here only the basic processes will be outlined. This will identify the minimum types of data needed to model watershed runoff processes.

Figure 11.2 provides a schematic of the hydrological cycle, identifying the processes that are either inputs, such as precipitation and air temperature, and those that are modelled, such as snowmelt or freezing, infiltration, transpiration, percolation, evaporation, and surface and groundwater runoff.

The surface water runoff from watershed land surfaces is affected by many factors. Among them are the amounts and intensity of precipitation and its rain and snow or ice content, the land cover, soil type and slope, and the

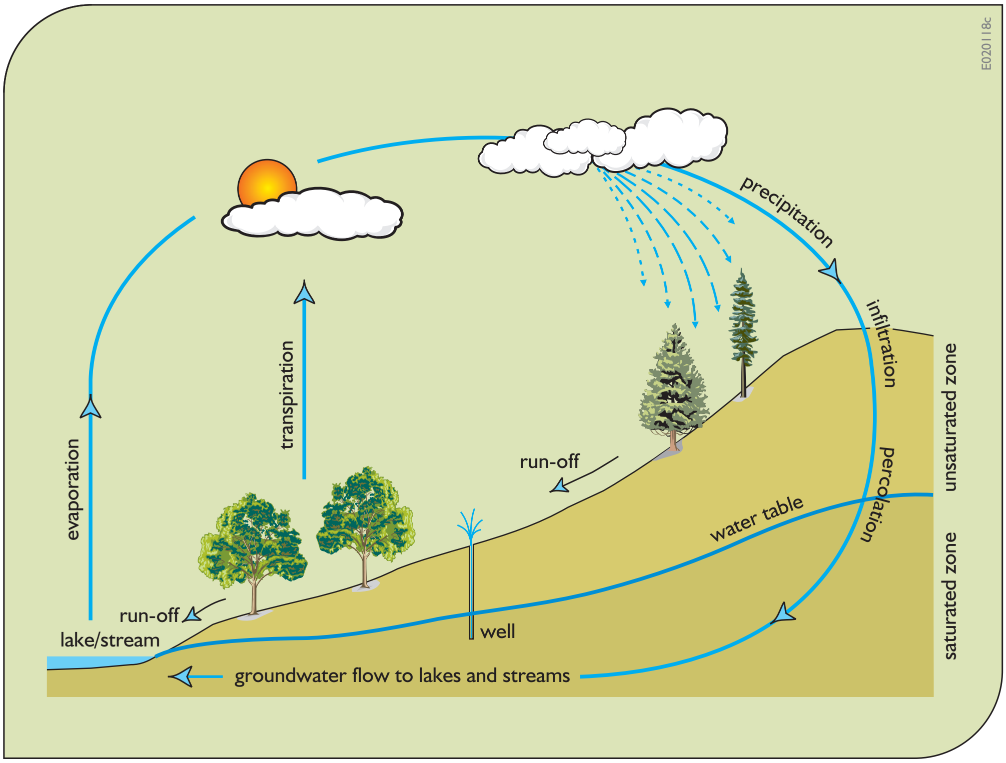

Figure 11.2. A schematic diagram of the hydrological cycle applicable to a watershed.

infiltration, evaporation and transpiration that take place. Infiltration, evaporation and transpiration rates are dependent on the land cover, soil type, soil moisture content and air temperature. Both the erosion that takes place and the sediment in the runoff depend on the intensity of precipitation, the land cover, the soil type and the slope. The concentration of chemical constituents in the runoff will depend on the accumulation of chemicals on the land at the time of a storm, the extent they dissolve in the surface water and the extent they are attached to the soil particles that are contained in the runoff. Here, we focus only on the runoff itself, as depicted in Figure 11.2.

The groundwater contribution of runoff to surface water bodies will depend on the relationship between the groundwater and the surface water body. Groundwater runoff can continue long after the last storm may have ended.

Rainfall runoff processes are complex, and depend on the particular physical and biological characteristics of each watershed. One of the difficulties in developing accurate predictive models of rainfall–runoff relationships is the considerable amount of data one must collect to understand just what processes are taking place at rather small spatial and temporal scales and then quantify these processes. Very often such data are just not available, yet we still need some way to predict, at least in some relative sense, the quantities and qualities of surface runoff associated with alternative land use policies or practices and precipitation events.

Ideally, one would like to have continuous records of net precipitation and air temperature for each watershed or sub-watershed being modelled. Usually the hydrological and meteorological data generally available are daily averages. Thus most continuous-time rainfall–runoff models use daily time steps, accepting that this does not capture the intensities of shorter duration storms during the day. A half-hour cloud burst from a thunderstorm over a portion of a modelled watershed may generate much more surface runoff (and sediment loads) than the same amount of precipitation evenly distributed over the entire watershed being modelled over a twenty-four-hour period. The use of daily averages over larger areas will not predict runoff accurately, but they are often the most detailed data one can expect to obtain in practice. More detailed rainfall data can sometimes be obtained from radar images, but the processes of calibrating and analysing such images are relatively expensive.

Precipitation

Precipitation can be in liquid and/or frozen form, depending of course on the location and time of year. In this discussion the term rain will denote liquid precipitation and snow will denote frozen precipitation (which could include ice as well). The proportion of rain and snow in precipitation will depend on the temperature of the air through which the precipitation falls. The proportion of rain and snow, if not directly measured, usually has to be estimated from average daily surface air temperature data, since these are the most detailed data available. Clearly air temperatures will vary over a day, as well as with altitude. The range of sub-freezing air temperatures over time and at different altitudes can affect the proportion of snow and ice in precipitation as well as the fraction of water content in the snow and ice. Estimates of these proportions, often based on average daily surface temperatures at one or at best only a few measuring stations, are sources of error when direct measurements of snow depth (water equivalent) and rain are not available.

The total net depth of precipitation, P_t, in day t, equals the sum of the net rain R_t and snow S_t (water equivalent) depths that reach the ground. These are input data for watershed runoff models.

Surface Water

The water available for surface runoff during any day is the free water already on the ground surface plus the rain and the snowmelt less any losses or reductions due to infiltration and evaporation and any freezing of standing surface water. The depth of snow that melts or the amount of freezing of free-standing water on the ground's surface will depend on the depth of snow available for melting, or on the depth of free surface water available for freezing, and on the air temperature.

Total snow depth accumulation, measured in water content, is determined from a mass balance of snow in the watershed. The initial snow depth, SD_t, at the beginning of day t plus the added depth of snow precipitation, S_t, and the depth of surface water that freezes on the surface, WF_t, less the snowmelt, SM_t, equals the final snow depth, SD_{t+1}, at the end of each day t:

$$SD_{t+1} = SD_t + S_t + WF_t - SM_t \tag{11.1}$$

The actual daily snowmelt, SM_t, will be the maximum snowmelt associated with the average air temperature of that day, $SM_t^{\max}$, or the actual amount of snow available for melting, $SD_t + S_t$:

$$SM_t = \min\{SM_t^{\max}, SD_t + S_t\} \tag{11.2}$$

Similarly, for the freezing of surface water, SW_t and rainfall R_t,

$$WF_t = \min\{WF_t^{\max}, SW_t + R_t\} \tag{11.3}$$

One would not expect melting and freezing to occur on the same day in a model based on average daily temperatures.

The net amount of water available for runoff, AW_t, before other losses are considered, equals the depth available at the beginning of the day, SW_t, plus the rainfall depth, R_t, and snowmelt depth, SM_t, less the freezing depth, WF_t:

$$AW_t = SW_t + R_t + SM_t - WF_t \tag{11.4}$$

This amount of water is reduced by that which infiltrates into the unsaturated soil, if any, and by that which evaporates.

Infiltration and Percolation

Infiltration may occur if the ground is not frozen or completely saturated. The ground is usually considered frozen if snow is on top of it (that is, the accumulated snow depth is greater than zero). It could also be frozen after several days of air temperatures below 0°, even if no snow depth exists. In these cases infiltration is assumed to be zero. If infiltration can occur, the amount will depend on the soil cover, type and moisture content, as well as on the available water to infiltrate if that amount is less than the maximum infiltration rate over a day. Different soil types have different effective porosities (fraction of total soil volume occupied by pores that water can enter), and different maximum rates of infiltration depending, in part, on the soil moisture content.

The maximum soil moisture content of unsaturated soil is reached when its available pore space is occupied by water. This amount, expressed as depth, is equal to the effective porosity of the soil times the depth of the unsaturated zone, UD_t. Expressing soil moisture content variable, MC_t, as a fraction of the soil's maximum capacity,

Depth of soil moisture at beginning of day t,

$$= MC_t(\text{effective porosity})(UD_t) \tag{11.5}$$

The average depth of the unsaturated zone, UD_t, approximates the difference between the average watershed surface elevation, WE, and the average unconfined groundwater table elevation, GE_t:

$$UD_t = WE - GE_t \tag{11.6}$$

The infiltration depth, I_t, occurring when the ground is not frozen and free of snow depth, will be the maximum rate, I^{max}, for the soil type, the soil moisture deficit, or the water depth available for infiltration, AW_t, whichever is less.

$$\begin{aligned} I_t &= \min\{AW_t, I^{max}, [(\text{effective porosity})(UD_t)(1-MC_t)]\} \\ &\quad \text{if soil not frozen and } SD_t = 0 \\ &= 0 \text{ otherwise} \end{aligned} \tag{11.7}$$

The depth of percolation of water, UP_t, in the unsaturated soil zone to the groundwater aquifer is assumed to equal either the maximum daily percolation depth, P^{max}, for the soil type or the available moisture content down to some minimum percentage, P^{min}, of the maximum amount, whichever is less. Often P^{max} is assumed to equal the maximum infiltration rate, I^{max}, for lack of any better assumption or measured data.

$$\begin{aligned} UP_t = \min\{&P^{max}, \max[0, (MC_t - P^{min}) \\ &\times(\text{effective porosity})(UD_t)]\} \end{aligned} \tag{11.8}$$

Transpiration and Evaporation

Transpiration depends on the moisture content in the unsaturated soil, the land cover, the depth of plant roots and the air temperature. Land cover and root depth may vary over a year.

Plant transpiration, T_t, reduces the soil moisture content of the unsaturated zone if the depth of that zone, UD_t, exceeds the average root zone depth, RZ_t. If the unsaturated zone depth is less than the average root zone depth, then roots have access to the groundwater and the transpiration will reduce the groundwater table. In this case, the transpiration from groundwater, TG_t, is usually assumed equal to the maximum transpiration rate, T_t^{max}.

Some watershed rainfall–runoff models keep track of the root depths, especially in agricultural croplands, and their impact on soil moisture and transpiration. Alternatively, one can assume transpiration will equal its maximum rate, T_t^{max}, for the given land cover and air temperature until the soil moisture content fraction, MC_t, goes below some minimal moisture content fraction for transpiration, MCT^{min}. Transpiration will cease when soil moisture contents go below that minimum percentage. This water is not available to the plants.

$$\begin{aligned} T_t = \min\{&T_t^{max}, \max[0, (MC_t - MCT^{min}) \\ &\times(\text{effective porosity})(UD_t) - UP_t\}] \end{aligned}$$

and

$$TG_t \text{ is 0 if } UD_t \geq RZ_t \tag{11.9}$$

otherwise

$$TG_t = T_t^{max} \text{ and } T_t \text{ is 0} \quad \text{if } UD_t < RZ_t \tag{11.10}$$

The depth of evaporation of water on the land surface is a function of the air temperature, relative humidity and wind velocities. Maximum daily evaporation depths are usually assumed to be fixed values that depend on average meteorological conditions for various seasons of the year when more precise data or functions defining evaporation are not available or not used. The actual evaporation each day will be the maximum evaporation rate for that day, E_t^{max}, or the total depth of free surface water, AW_t, less infiltration, I_t, whichever is less.

$$E_t = \min\{AW_t - I_t, E_t^{max}\} \tag{11.11}$$

Surface Runoff

The depth of surface water available for runoff, SA_t, in each day t equals the depth of free surface water, AW_t, (which results from the initial amount, SW_t, plus rain, R_t, and snowmelt, SM_t, less the amount that freezes, WF_t, as defined in Equation vation. 11.4) less the infiltration I_t, and evaporation, E_t:

$$SA_t = AW_t - I_t - E_t \tag{11.12}$$

The depth of surface runoff, SR_t, is some fraction of this amount, depending on the slope, the extent of ponding, the surface area and the land cover of the watershed.

Surface runoff can be estimated in a number of ways. (See, for example, the curve number method of USDA, 1972, as presented in Chapter 13). For this discussion, assume Manning's Equation applies. This is an equation for determining the velocity of overland and channel flow based on the hydraulic radius, HR_t, (cross-sectional area of flow divided by the wetted perimeter), land slope and ground cover. For overland sheet flow, and for channel flow of small depths, the hydraulic radius is essentially the depth of available surface water, SA_t. If SA_t is in units of metres, the velocity, V_t, of surface runoff (metres/second) based on Manning's equation, equals

$$V_t = (1/n)(SA_t)^{2/3}(\text{Slope})^{1/2} \quad (11.13)$$

The parameter n represents an effective roughness coefficient that includes the effect of raindrop impact, drag over the ground surface from obstacles such as litter, crop ridges and rocks, and erosion and transportation of sediments. This roughness coefficient will be the major calibration parameter of the runoff model.

Watershed runoff will be a combination of sheet flow and channel flow. Typical n values for sheet flow range from 0.011 for smooth paved surfaces to 0.1–0.4 for grass to 0.4–0.8 for underbrush in wooded areas. These n values are for very shallow flow depths of about 20–50 millimetres or so. For river channels the n values typically range between 0.02 and 0.07 (USDA-SCS, 1972).

The maximum surface water flow, Q_t^{max}, equals the velocity times the cross-sectional area of the flow perpendicular to the direction of flow. The cross-sectional area is the depth of available water, SA_t, times the average width, W, of the watershed. If the units of width are expressed in metres, the maximum flow in cubic metres per second is

$$Q_t^{max} = (1/n)(SA_t)^{5/3}(\text{Slope})^{1/2}W \quad (11.14)$$

The total quantity of surface water runoff, SR_t, from the watershed in day t will be this maximum flow rate multiplied by the number of seconds in a day or the total amount of water available for runoff, whichever is less.

$$SR_t = \min\{Q_t^{max}(60)(60)(24),(SA_t)(\text{Watershed area})\} \quad (11.15)$$

In reality, most watersheds are not uniform, flat, tilted landscapes where sheet flow takes place. The water available for runoff will either seep into the upper soil layer and reappear down slope or flow into small channels. The above equations based on average watershed conditions are thus a surrogate for what actually takes place at much smaller space scales than are reasonable to model.

2.1.3. Hydrological Processes: Groundwater

Groundwater Runoff to Streams and Rivers

To estimate the groundwater contribution of the total runoff, it is necessary to model the surface water–groundwater interaction. As shown in Figure 11.3, groundwater can move along flow paths of varying lengths from areas of recharge to areas of discharge. The generalized flow paths in Figure 11.3 start at the water table of the upper unconfined aquifer and continue through the groundwater system, terminating at the surface water body. In the uppermost, unconfined aquifer, flow paths near the

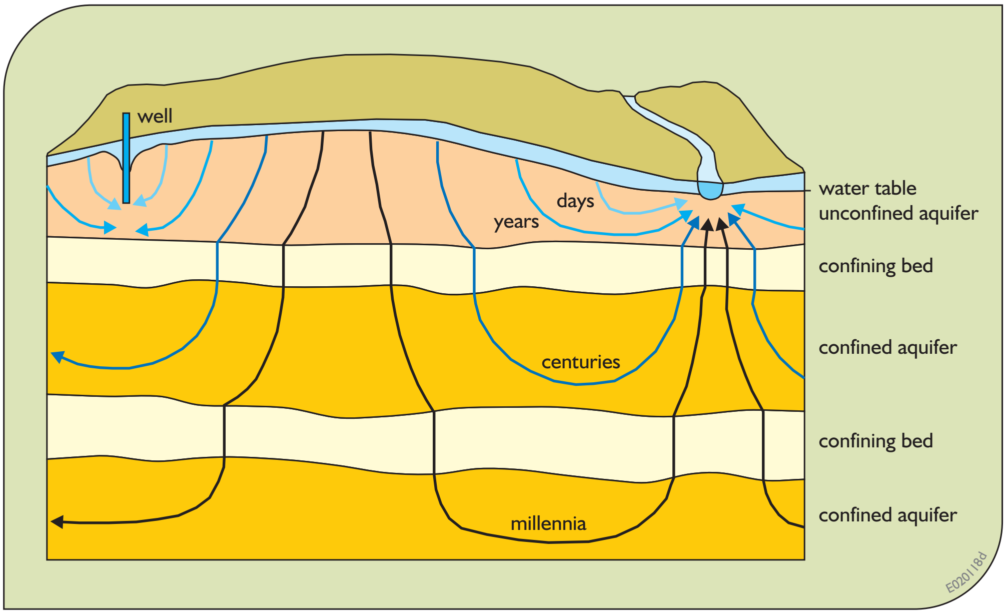

Figure 11.3. Groundwater–surface water interactions over a range of time and space scales.

stream can be tens to hundreds of metres in length and have corresponding travel times ranging from days to several years. The longest and deepest flow paths in Figure 11.3 may be thousands of kilometres in length, and travel times may range from decades to millennia. In general, shallow groundwater is more susceptible to contamination from human sources and activities because of its close proximity to the land surface.

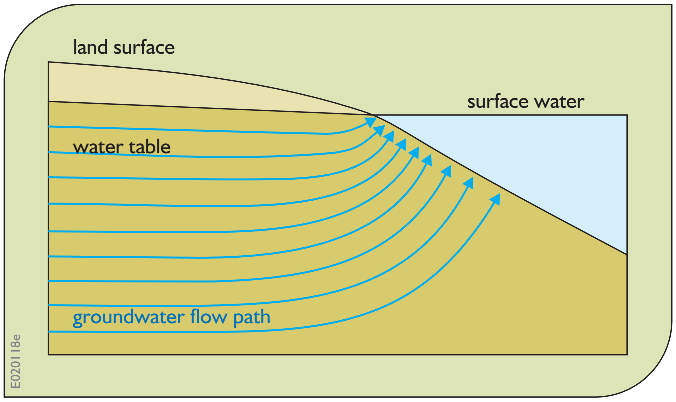

Figure 11.4. Groundwater seepage into surface water usually is greatest near shore. In flow diagrams such as that shown here, the quantity of discharge is equal between any two flow lines; therefore, the closer flow lines indicate greater discharge per unit of bottom area.

Streams interact with groundwater in three basic ways. They gain water from inflow of groundwater through the streambed (gaining stream, Figure 11.5A), they lose water to groundwater by outflow through the streambed (losing stream, Figure 11.6A), or they do both, gaining in some reaches and losing in others. For groundwater to flow into a stream channel, the elevation of the groundwater table in the vicinity of the stream must be higher than that of the stream-water surface. Conversely, for surface water to seep to groundwater, the elevation of the water table in the vicinity of the stream must be lower than that of the stream-water surface. Contours of water-table elevation indicate gaining streams by pointing in an upstream direction (Figure 11.5B), and indicate losing streams by pointing in a downstream direction (Figure 11.6B) in the immediate vicinity of the stream.

Losing streams can be connected to the groundwater system by a continuous saturated zone (Figure 11.6A) or can be isolated from the groundwater system by an

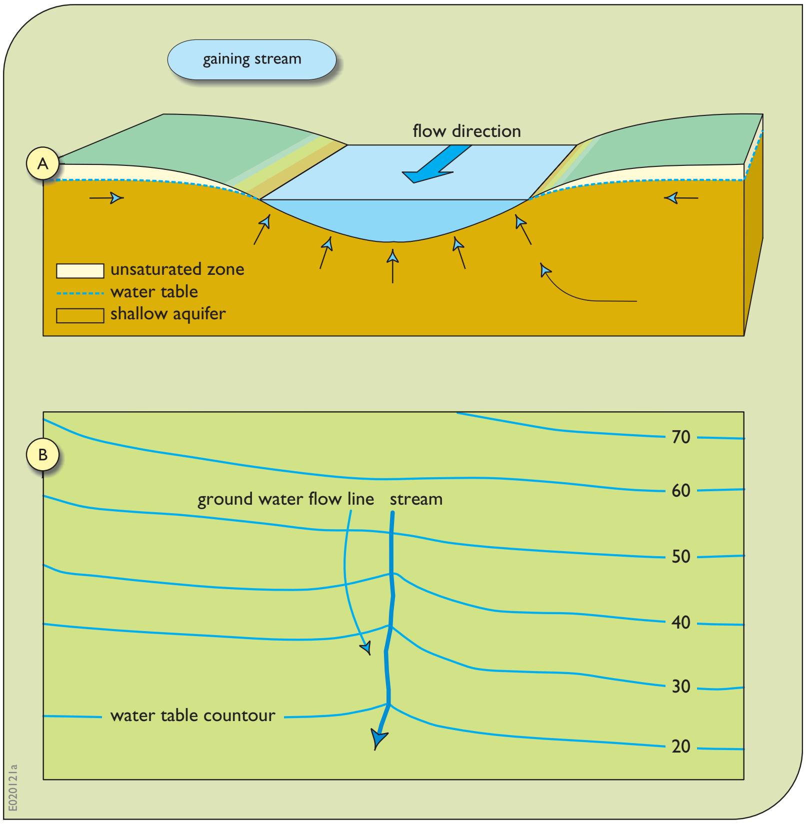

Figure 11.5. Gaining streams receive water from the groundwater system (A). This can be determined from water-table contour maps because the contour lines point in the upstream direction where they cross the stream (B).

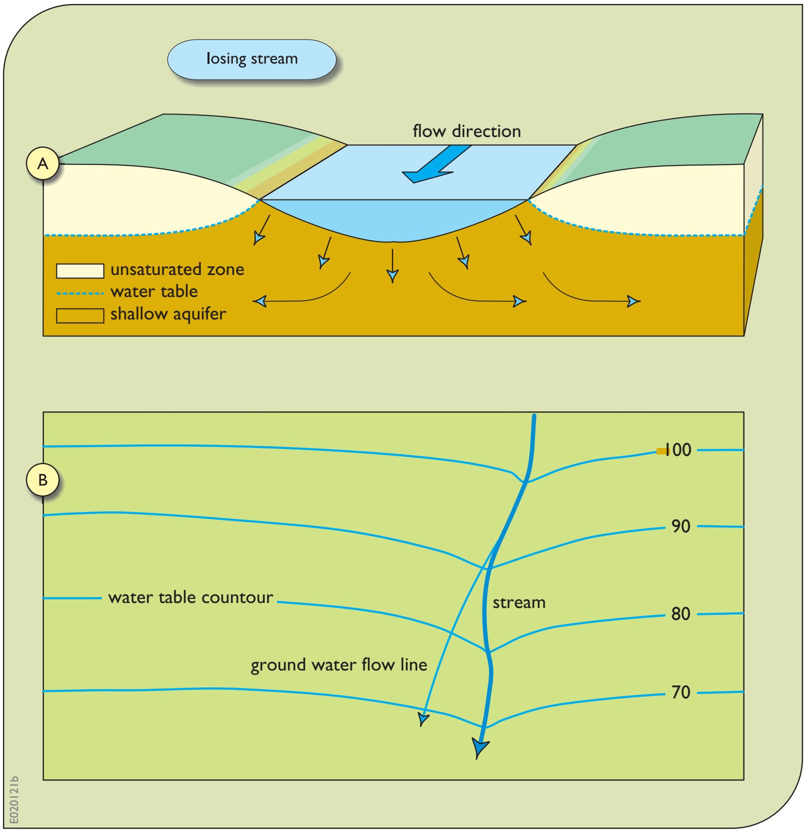

Figure 11.6. Losing streams lose water to the groundwater system (A). This can be determined from water-table contour maps because the contour lines point in the downstream direction where they cross the stream (B).

unsaturated zone. In the latter case, infiltration and percolation processes apply. Pumping of shallow groundwaters near disconnected streams does not affect the flow of water in those streams near the pumped wells.

Given the complexity of the interactions between surface water and groundwater at specific sites under different conditions, it becomes difficult to model these interactions without considerable site-specific data based on many detailed observations and measurements. As a first approximation, Darcy's equation for laminar (slowly moving) flow in saturated porous soils can be used to estimate the flow either from the surface water to the groundwater aquifer, or vice versa. This flow depends on the hydraulic conductivity of the soil, K (having dimensions of length per unit time, L/T), and the area through which the flow occurs, A (L^2). It also depends on the groundwater flow head gradient, dH/dX (the change in groundwater head H per change in distance X perpendicular to the stream or river reach). Darcy's equation states that the groundwater flow is the product of $KA(dH/dX)$.

Let $(dH/dX)_L$ and $(dH/dX)_R$ be the groundwater head gradients in the direction of groundwater flow on the left and right side of the stream or river reach, respectively. Using Darcy's equation, the net flow, Q_{sg}, from surface water stream or river reach to the unconfined groundwater aquifer is the sum of the products of the hydraulic conductivity K, the area A, and the groundwater head gradient, dH/dX, for each side of the stream or river reach.

$$Q_{sg} = K_L A_L (dH/dX)_L - K_R A_R (dH/dX)_R \qquad (11.16)$$

If Q_{sg} is negative, the net flow is from the groundwater aquifer to the surface water reach. Clearly, no more water can flow from one water body to another than the volume of water in the former. Models that use equations such as Equation 11.16 need to check on this condition.

Groundwater Runoff to Lakes, Reservoirs and Wetlands

Surface water in lakes, wetlands and reservoirs interacts with groundwater rather the same way that streams and rivers do, as illustrated in Figure 11.7. Although the basic interactions are the same for these storage water bodies as they are for streams, however, the specific interactions differ in several ways.

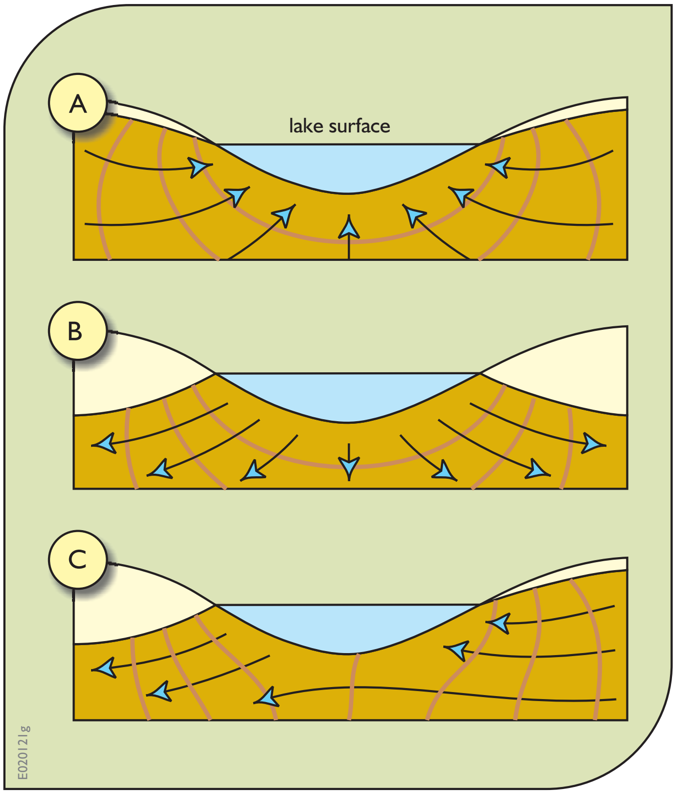

Figure 11.7. Lakes can receive groundwater inflow (A), lose water as seepage to groundwater (B), or both (C).

The water levels of natural lakes are not controlled by dams. Hence they generally do not change as rapidly as the water levels of streams, and bank storage is therefore of lesser importance in lakes than in streams. Furthermore, lake sediments commonly have greater volumes of organic deposits. These less permeable organic deposits reduce the seepage and biogeochemical exchanges of water and solutes that take place in lakes compared with those which take place in streams.

Reservoirs are human-made lakes that are designed primarily to control the flow and distribution of surface water. Most reservoirs are constructed in stream valleys, and therefore have some characteristics of both streams and lakes. Like streams, reservoirs can have widely fluctuating levels, bank storage can be significant, and they commonly have a continuous flushing of water through them.

Wetlands that occupy depressions in the land surface have interactions with groundwater similar to lakes and streams. They can receive groundwater inflow, recharge groundwater, or do both. Unlike streams and lakes, however, wetlands do not always occupy low points and depressions in the landscape (Figure 11.8A). They can be present on slopes (fens, for example) or even on drainage divides (some types of bogs). Fens are wetlands that commonly receive most of their water from groundwater discharge (Figure 11.8B). Bogs are wetlands that occupy uplands (Figure 11.8D) or extensive flat areas, and they receive much of their water from precipitation. While riverine wetlands (Figure 11.8C) commonly receive groundwater discharge, they are dependent primarily on the stream for their water supply.

A major difference between lakes and wetlands, with respect to their interaction with groundwater, is the ease with which water moves through their beds. Lakes are commonly shallow around their perimeter, where waves can remove fine-grained sediments, permitting the surface water and groundwater to interact freely. Wetlands, on the other hand, typically contain fine-grained and highly decomposed organic sediments near the wetland edge, slowing the transfer of water and solutes between groundwater and surface water.

Another difference in the interaction between groundwater and surface water in wetlands compared with lakes is the extent of rooted vegetation in wetlands. The fibrous root mat in wetland soils is highly conductive to water flow; therefore, water uptake by roots of emergent plants results in significant interchange between surface water and pore water of wetland sediments. Water exchanges in this upper soil zone are usually not as restricted as the exchanges between surface water and groundwater at the base of the wetland sediments.

All these complex interactions between ground and surface water systems can only be approximated by watershed models, even the most detailed ones. Without extensive data, it is unlikely the models will capture all the interactions that take place over space and time.

2.1.4. Modelling Groundwater: Surface Water Interactions

One of the simplest ways of estimating the groundwater runoff from a watershed is to assume the runoff will equal some fraction of the average groundwater head. This fraction is called a *recession constant*.

If the aquifer lies solely under the watershed of interest, then a recession rate can be defined for estimating

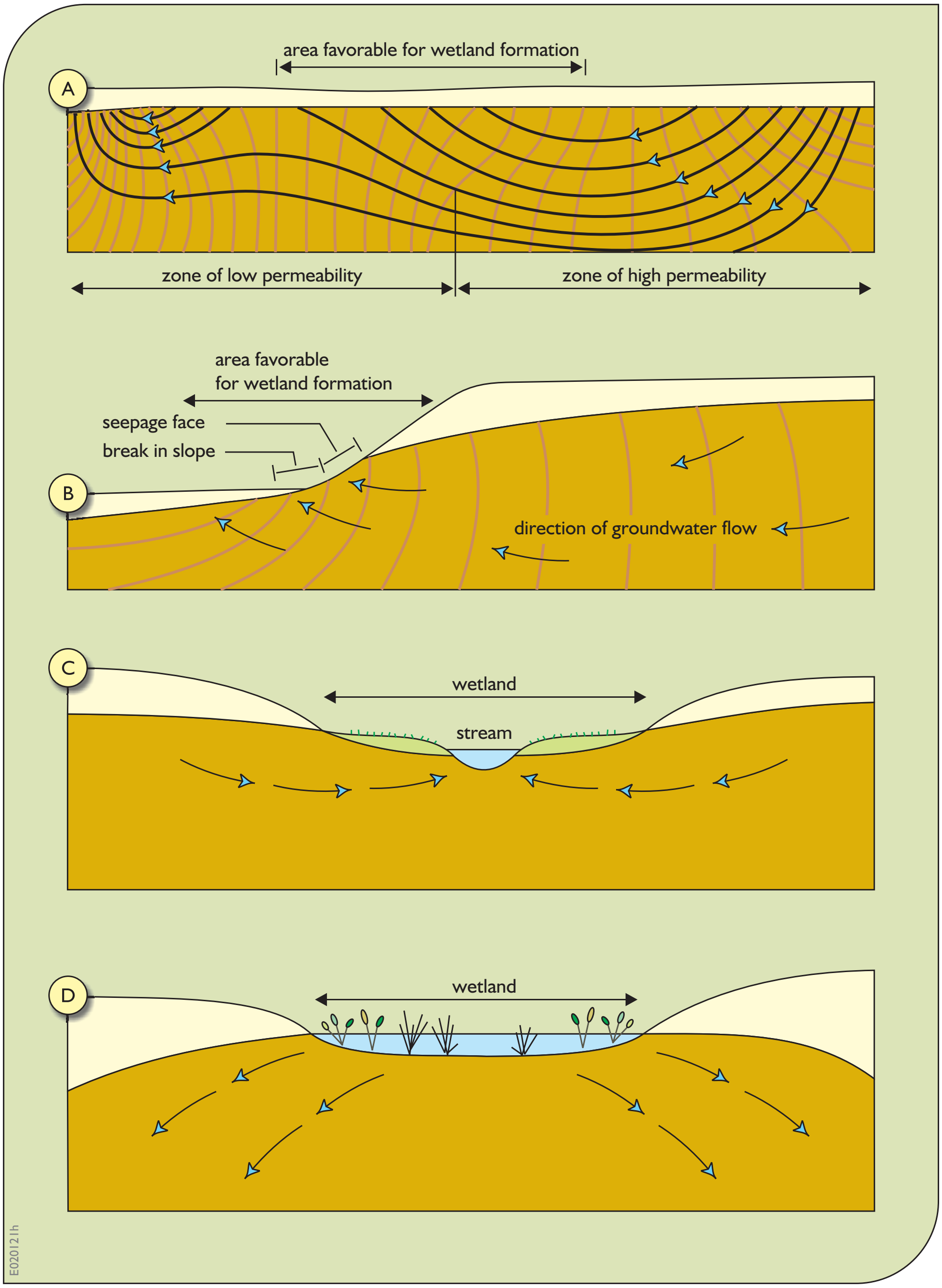

Figure 11.8. The source of water to wetlands can be from groundwater discharge where the (light-coloured) land surface is underlain by complex groundwater flow fields (A), from groundwater discharge at seepage faces and at breaks in slope of the water table (B), from streams (C), and from precipitation in cases where wetlands have no stream inflow and groundwater gradients slope away from the wetland (D).

the extent of groundwater base flow depth when the average groundwater table elevation, GE_t, is greater than the average surface water body elevation, SE_t. Letting the dimensionless recession fraction be k, the groundwater base flow contribution, GR_t, in day t, to total surface runoff is

$$GR_t = \text{Max}\{0, (GE_t - SE_t\}k(\text{aquifer area}) \times (\text{effective aquifer porosity}) \quad (11.17)$$

The total runoff in day t equals the sum of the groundwater runoff, GR_t, plus the surface water runoff SR_t.

This method does not consider situations in which water flows from the surface water bodies to the groundwater aquifer.

Alternatively, the two-way interactions between the ground and surface waters, perhaps involving multiple watersheds and one or more underlying unconfined aquifers, can be modelled using Darcy's equation (Equation 11.17).

If the groundwater aquifer is not being modelled in detail, all one may have available from a lumped watershed model is the average groundwater table elevation or head at the beginning or end of any time period. This average head is usually based on the groundwater volume. For an unconfined aquifer, the average head, H_t, at the beginning of day t, will equal the groundwater volume, GWV_t, divided by the product of the aquifer area times its effective porosity. The head gradient is the difference between the average groundwater elevation (head), GE_t, and the surface elevation, SE_t, of the surface water body, divided by some representative distance ΔX.

$$-\,\mathrm{d}H_t/\mathrm{d}X \approx [GE_t - SE_t]/\Delta X \tag{11.18}$$

Many lumped models of groundwater aquifers approximate the head gradient by dividing the elevation difference between the average groundwater table elevation, GE_t, and the surface water body elevation, SE_t, by no more than a quarter of the distance from the surface water body and the aquifer boundary. This is shown in Figure 11.9 for water-gaining surface water bodies. In this figure, the ΔW term is equivalent to the ΔX terms use in Equations 11.18 and 11.19.

The approximate area, A_t, through which the groundwater flows is the average of the surface water body elevation above the base of the aquifer, SE_t, and the average groundwater head elevation, GE_t, times the length, L, of the surface water body in its direction of flow. Assuming groundwater flows from both sides of the river into the river and the K and ΔX (or ΔW in Figure 11.9) terms are the same for both sides of the river, Darcy's equation for

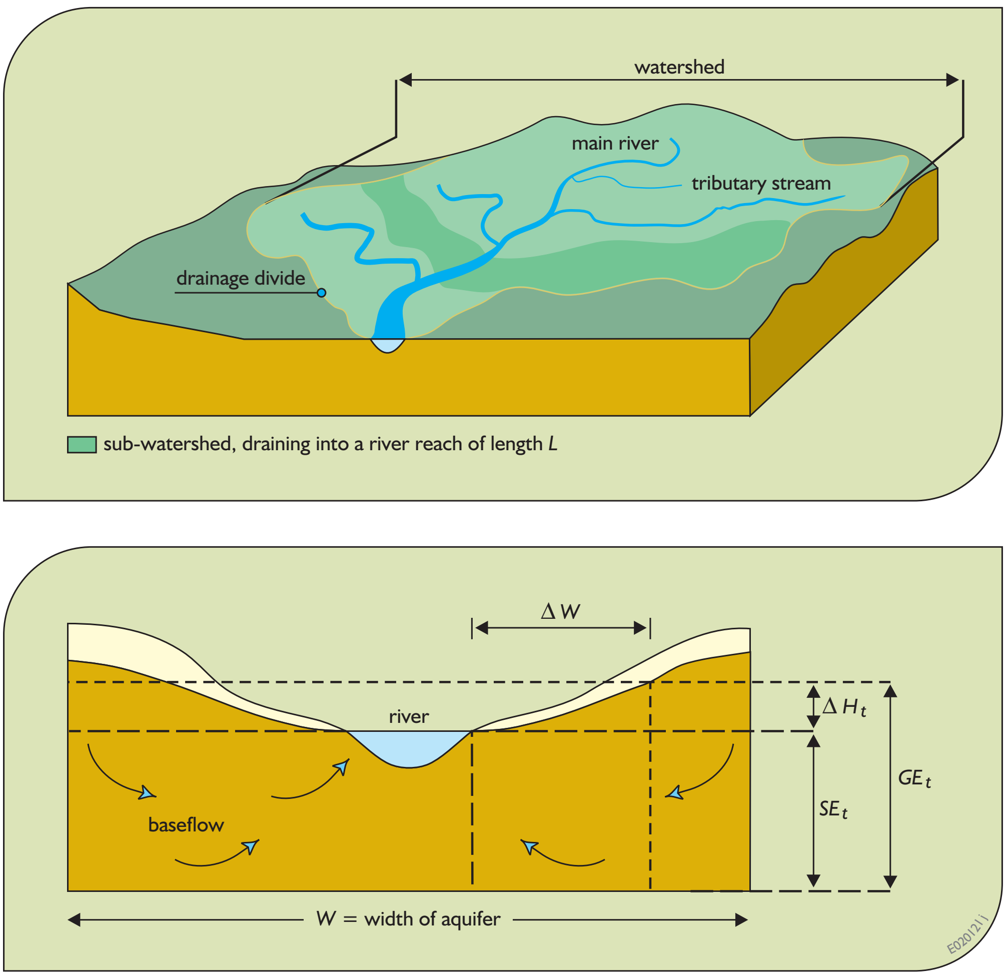

Figure 11.9. Cross-sectional dimensions of a watershed showing a surface water body that is receiving water from the groundwater aquifer.

estimating the groundwater runoff, GR_t from this portion of the watershed is:

$$GR_t = 2K((SE_t + GE_t)/2)(L)[GE_t - SE_t]/\Delta X$$
$$= K(GE_t^2 - SE_t^2)(L)/\Delta X \quad (11.19)$$

This equation can apply to any surface water body receiving runoff from groundwater. It can also apply to any groundwater aquifer receiving water from a surface water body. In this case, GR_t would be negative. As for Equation 11.16, no more water can flow from one water body to another than the volume of water in the former. Again, models that use equations such as Equation 11.19 need to check on this condition.

2.1.5. Streamflow Estimation

Water resources managers need to have estimates of streamflows at each site where management decisions are to be made. These streamflows can be based on the results of rainfall runoff models or on measured historical flows at stream gauges. For modelling alternative management policies, these streamflows should be those that would have occurred under natural conditions. These are called *naturalized flows*. These naturalized flows are often computed from observed gauge flows adjusted to take into account any upstream regulation and diversions. Many gauge flow measurements reflect actions taken upstream that alter the natural flows, such as diversions and reservoir storage. Unless such upstream water management and use policies are to continue, these regulated flows should be converted to unregulated or natural flows prior to their use in management models.

Alternatively, naturalized flows can be estimated from rainfall–runoff modelling.

Rainfall–Runoff Modelling

In rainfall–runoff models, mass balances must be maintained for snow depth, for free surface water depth, for the moisture content of the unsaturated zone, and for the groundwater volume, as applicable.

For surface water, the remaining free surface water will be that water which is left at the end of period t. The free water remaining on the watershed's surface at the end of day t, SW_{t+1}, is what exists at the beginning of day $t+1$. It equals the surface water that exists at the beginning of the day, SW_t, plus the rainfall, R_t, plus the snowmelt, SM_t, less the depth that freezes, WF_t, the infiltration, I_t, the evaporation, E_t, and the depth of surface runoff, SR_t/(watershed area):

$$SW_{t+1} = SW_t + R_t + SM_t - WF_t - I_t - E_t$$
$$- SR_t/(\text{watershed area})$$
$$= SA_t - SR_t/(\text{watershed area}) \quad (11.20)$$

The snow depth, SD_{t+1}, at the end of each day t is:

$$SD_{t+1} = SD_t + S_t + WF_t - SM_t \quad (11.21)$$

The final groundwater volume is equal to the initial groundwater volume, GWV_t, plus any additions and less any reductions during the day. These additions and reductions include the total net runoff to surface water bodies, GR_t, the percolation, UP_t, the groundwater transpiration, TG_t, any groundwater abstractions or artificial recharge, less any losses to, GWQ_t^{out}, or plus any gains from, GWQ_t^{in}, deeper aquifers.

$$GWV_{t+1} = GWV_t + \left[\sum_{\text{watersheds}} GR_t + (UP_t - TG_t) \times (\text{watershed area})\right] + GWQ_t^{\text{in}} - GWQ_t^{\text{out}} \quad (11.22)$$

The average groundwater table elevation is computed taking into account the percolation from the unsaturated zone and its inflow from or outflow to the surface water system. The groundwater head or elevation is the watershed elevation or its volume divided by its area times its porosity, whichever is less:

$$GE_{t+1} = \min\{WE, \{GWV_{t+1}/[(\text{aquifer area}) \times (\text{effective porosity})]\}\} \quad (11.23)$$

The unsaturated zone depth at the end of each day t is

$$UD_{t+1} = WE - GE_{t+1} \quad (11.24)$$

The soil moisture fraction, MC_{t+1}, in the unsaturated soil layer at the end of each day can be calculated as

$$MC_{t+1} = [MC_t(\text{porosity})(UD_t) + I_t - T_t - UP_t]/(\text{effective porosity})(UD_{t+1}) \quad (11.25)$$

If the groundwater table is rising, the actual soil moisture content will probably increase due to capillary action. If the groundwater table is falling, the actual soil moisture content is likely to be higher than estimated due to the delayed percolation of the excess water from the once-saturated zone. Here, we assume water table elevation changes (that is, unsaturated zone depth changes) do not by themselves change the percentage of soil moisture in the unsaturated zone.

If the groundwater table equals the surface elevation, then the unsaturated zone, *UD*, does not exist and no infiltration and percolation will occur. In this case, the unconfined saturated groundwater table is at the surface.

Streamflow Estimation Based on Flow Data

Assuming unregulated streamflow data are available at gauge sites, these sites may not be where flow data are needed. Thus, some way is needed to generate corresponding streamflow data at sites where managers need them, such as at potential diversion or reservoir sites. Just how this can be done depends in part on the durations of a model's time periods.

Consider, for example, the simple river basin illustrated in Figure 11.10. The streamflows have been recorded over a number of years at gauge sites 1 and 9. Knowledge of the flows Q_t^s in each period t at gauge sites $s = 1$ and 9 permits the estimation of flows at any other site in the basin as well as the incremental flows between those sites in each period t.

The method used to estimate flows at ungauged sites will depend on the characteristics of the watershed of the river basin. In humid regions where streamflows increase in the downstream direction, and the spatial distribution of average monthly or seasonal rainfall is more or less the same from one part of the river basin to another, the runoff per unit land area is assumed constant over space. In these situations, estimated flows, q_t^s, at any site s are usually based on the watershed areas, A^s, contributing flow to those sites, and the corresponding streamflows and watershed areas above the nearest or most representative gauge sites.

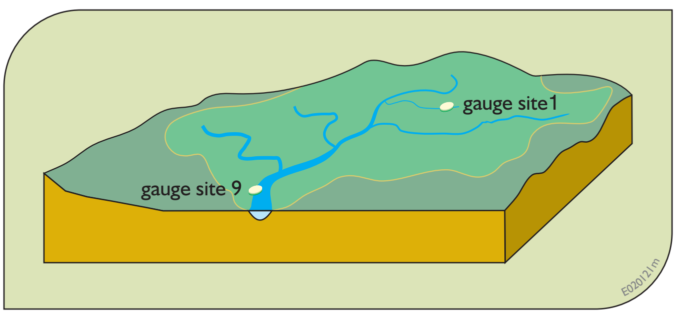

Figure 11.10. River basin gauge sites where streamflows are measured and recorded.

For each gauge site, the runoff per unit land area can be calculated by dividing the gauge flow, Q_t^s, by the upstream drainage area, A^s. This can be done for each gauge site in the basin. Thus for any gauge site g, the runoff per unit drainage area in month or season t is Q_t^g divided by A^g. This runoff per unit land area times the drainage area upstream of any site s of interest will be the estimated streamflow in that period at that site s. If there are multiple gauge sites, as illustrated in Figure 11.10, the estimated streamflow at some ungauged site s can be a weighted linear combination of those unit area runoffs times the area contributing to the flow at site s. The non-negative weights, w_g, that sum to 1 reflect the relative significance of each gauge site with respect to site s. Their values will be based on the judgements of those who are familiar with the basin's hydrology.

$$Q_t^s = \left\{ \sum_g w_g Q_t^g / A^g \right\} A^s \qquad (11.26)$$

In all the models developed and discussed below, the variable Q_t^s will refer to the mean natural (unregulated) flow (L^3/T) at a site s in a period t.

The difference between the natural streamflows at any two sites is called the *incremental flow*. Using Equation 11.26 to estimate streamflows will result in positive incremental flows. The downstream flow will be greater than the upstream flow. In arid regions, incremental flows may not exist and hence, due to losses, the flows may be decreasing in the downstream direction. In these cases, there is a net loss in flow in the downstream direction. This might be the case when a stream originates in a wet area and flows into a region that receives little if any rainfall. In such arid areas the runoff into the stream, if any, is less than the evapotranspiration and infiltration into the ground along the stream channel.

For stream channels where there exist relatively uniform conditions affecting water loss and where there are no known sites where the stream abruptly enters or exits the ground (as can occur in karst conditions), the average streamflow for a particular period t at site s can be based on the nearest or most representative gauge flow, Q_t^g, and a loss rate per unit length of the stream or river, L^{gs}, between gauge site g and an ungauged site s. If there are at least two gauge sites along the portion of the stream or river that is in the dry region, one can compute the loss of flow per unit stream length, and apply this loss rate to various sites along the stream or river. This loss rate per unit length, however, may not be constant over the entire length between the gauge stations, or even for all flow rates. Losses will probably increase with increasing flows simply because more water surface is exposed to evaporation and seepage. In these cases one can define a loss rate per unit length of stream or river as a function of the magnitude of flow.

In watersheds characterized by significant elevation changes, and consequently varying rainfall and runoff distributions, other methods may be required for estimating average streamflows at ungauged sites. The selection of the most appropriate method to use, as well as the most appropriate gauge sites to use, to estimate the streamflow, Q_t^s, at a particular site s can be a matter of judgement. The best gauge site need not necessarily be the nearest gauge to site s, but rather the site most similar with respect to important hydrological variables.

2.1.6. Streamflow Routing

If the duration of a within-year period is less than the time of flow throughout the stream or river system being modelled, and the flows vary within the system, then some type of streamflow routing must be used to keep track of where the varying amounts of water are in each time period. There are many proposed routing methods (as described in any hydrology text or handbook, e.g. Maidment, 1993). Many of these more traditional methods can be approximated with sufficient accuracy by relatively simple methods. Two such methods are described in the following paragraphs.

The outflow, O_t, from a reach of stream or river during a time period t is a function of the amount of water in that reach, i.e. its initial storage, S_t, and its inflow, I_t. Because of bank storage, that outflow is often dependent on whether the quantity of water in the reach is increasing or decreasing. If bank inflows and outflows are explicitly modelled, for example as described in Sections 2.1.3 and 2.1.4, or if bank storage is not that significant, then the outflow from a reach in any period t can be expressed as a simple two-parameter power function of the form $a(S_t + I_t)^b$. Mass-balance equations, which may take losses into account, update the initial storage volumes in each succeeding time period. The reach-dependent parameters a and b can be determined through calibration procedures such as genetic algorithms (Chapter 6), given a time series of reach inflows and outflows. The resulting outflow function is typically concave (the parameter b will be less than 1), and thus the minimum value of $S_t + I_t$ must be at least 1. If, due to evaporation or other losses, the reach volume drops below this or any pre-selected higher amount, then the outflow is assumed to be 0.

Alternatively, one can adopt a three- or four-parameter routing approach that fits a wider range of conditions. Each stream or river reach can be divided into a number of segments. That number n is one of the parameters to be determined. Each segment s can be modelled as a storage unit, having an initial storage volume, S_{st}, and an inflow, I_{st}. The three-parameter approach assumes the outflow, O_{st}, is a linear function of the initial storage volume and inflow:

$$O_{st} = \alpha S_{st} + \beta I_{st} \tag{11.27a}$$

Equation 11.27a applies for all time periods t and for all reach segments s in a particular reach. Different reaches may have different values of the parameters n, α and β. The calibrated values of α and β are non-negative and no greater than 1. Again, a mass-balance equation updates each segment's initial storage volume in the following time period. The outflow from each reach segment is the inflow into the succeeding reach segment.

The four-parameter approach assumes that the outflow, O_{st}, is a non-linear function of the initial storage volume and inflow:

$$O_{st}, = (\alpha S_{st} + \beta I_{st})^{\gamma} \tag{11.27b}$$

where the forth parameter γ is greater than 0 and no greater than 1. In practice, γ is very close to 1. Again the values of these parameters can be found using non-linear optimization methods, such as genetic algorithms, together with a time series of observed reach inflows and outflows.

Note the flexibility available when using the three- or four-parameter routing approach. Even blocks of flow can be routed a specified distance downstream over a specified time, regardless of the actual flow. This can be done

Box 11.1. Streamflow Routing in RIBASIM Simulation Model

Streamflow routing in RIBASIM is simulated with link storage nodes. The outflow from these 'nodes' (actually river segments acting as reservoirs) into the next node can be described by:

- Manning formula
- two-layered, multi-segmented Muskingum formula
- Puls method
- Laurenson non-linear 'lag and route' method.

by setting α and γ to 1, β to 0, and the number of segments n to the number of time periods it takes to travel that distance. This may not be very realistic, but there are some river basin reaches where managers believe this particular routing applies.

Most river basin models offer a number of alternative approaches to describe streamflow routing. Box 11.1 illustrates the methods included in the RIBASIM package (WL | Delft Hydraulics, 2004). Which method is the best to apply will depend on the required accuracy and the available data.

2.2. Lakes and Reservoirs

Lakes and reservoirs are sites in a basin where surface water storage needs to be modelled. Thus, variables defining the water volumes at those sites must be defined. Let S_t^s be the initial storage volume of a lake or reservoir at site s in period t. Omitting the site index s for the moment, the final storage volume in period t, S_{t+1}, (which is the same as the initial storage in the following period $t+1$) will equal the initial volume S_t plus the net surface and groundwater inflows, Q_t, less the release or discharge, R_t, and evaporation and seepage losses, L_t. All models of lakes and reservoirs include this mass-balance equation for each period t being modelled:

$$S_t + Q_t - R_t - L_t = S_{t+1} \tag{11.28a}$$

The release from a natural lake is a function of its surrounding topography and its water surface elevation. It is determined by nature, and unless it is made into a reservoir, its discharge or release is not controlled or managed. The release from a reservoir is controllable, and is usually a function of the reservoir storage volume and time of year. Reservoirs also have fixed storage capacities, K. In each period t, reservoir storage volumes, S_t, cannot exceed their storage capacities, K.

$$S_t \leq K \quad \text{for each period } t. \tag{11.28b}$$

Equations 11.28a and 11.28b are the two fundamental equations required when modelling water supply reservoirs. They apply for each period t.

The primary purpose of all reservoirs is to provide a means of regulating downstream surface water flows over time and space. Other purposes may include storage volume management for recreation and flood control, and storage and release management for hydropower production. Reservoirs are built to alter the natural spatial and temporal distribution of the streamflows. The capacity of a reservoir and its release (or operating) policy determine the extent to which surface water flows can be stored for later release.

The use of reservoirs for temporarily storing streamflows often results in a net loss of total streamflow due to increased evaporation and seepage. Reservoirs also bring with them changes in the ecology of a watershed and river system. They may also displace humans and human settlements. When considering new reservoirs, any benefits derived from regulation of water supplies, from floodwater storage, from hydroelectric power, and from any navigational and recreational activities should be compared to any ecological and social losses and costs. The benefits of reservoirs can be substantial, but so may the costs. Such comparisons of benefits and costs are always challenging because of the difficulty of expressing all such benefits and costs in a common metric. For a full discussion on the complex issues and choices on reservoir construction, refer to report of the World Commission on Dams (WCD, 2000).

Reservoir storage capacity can be divided among three major uses:

- *active storage* used for downstream flow regulation and for water supply, recreational development or hydropower production
- *dead storage* required for sediment collection
- *flood storage* capacity reserved to reduce potential downstream flood damage during flood events.

These separate storage capacities are illustrated in Figure 11.11. The distribution of active and flood control storage capacities may change over the year. For example

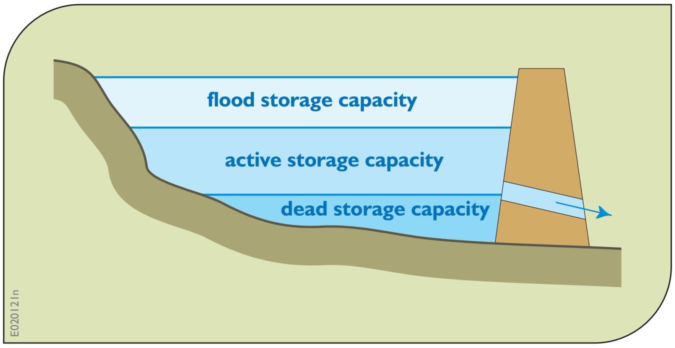

Figure 11.11. Total reservoir storage volume capacity consisting of the sum of dead storage, active storage and flood control storage capacities.

there is no need for flood control storage in seasons that are not going to experience floods.

Often these components of reservoir storage capacity can be modelled separately and then added together to determine total reservoir storage capacity. The next several sections of this chapter address how these capacities may be determined.

2.2.1. Estimating Active Storage Capacity

Mass Diagram Analyses

Perhaps one of the earliest methods used to calculate the active storage capacity required to meet a specified reservoir release, R_t, in a sequence of periods t, was developed by W. Rippl (1883). His *mass diagram analysis* is still used today by many planners. It involves finding the maximum positive cumulative difference between a sequence of pre-specified (desired) reservoir releases R_t and known inflows Q_t. One can visualize this as starting with a full reservoir, and going through a sequence of simulations in which the inflows and releases are added and subtracted from that initial storage volume value. Doing this over two cycles of the record of inflows will identify the maximum deficit volume associated with those inflows and releases. This is the required reservoir storage. Having this initial storage volume, the reservoir would always have enough water to meet the desired releases. However, this only works if the sum of all the desired releases does not exceed the sum of all the inflows over the same sequence of time periods. Reservoirs cannot make water. Indeed, they can lose it from evaporation and seepage.

Equation 11.29 represents this process. The active storage capacity, K_a, will equal the maximum accumulated storage deficit one can find over some interval of time within two successive record periods, T.

$$K_a = \text{maximum}\left[\sum_{t=i}^{j}(R_t - Q_t)\right] \qquad (11.29)$$

where $1 \le i \le j \le 2T$.

Equation 11.29 is the analytical equivalent of graphical procedures proposed by Rippl for finding the active storage requirements. Two of these graphical procedures are illustrated in Figures 11.12 and 11.13 for a nine-period inflow record of 1, 3, 3, 5, 8, 6, 7, 2 and 1. Rippl's original approach, shown in Figure 11.12, involves plotting the cumulative inflow $\sum_{\tau=1}^{t} Q_\tau$ versus time t. Assuming a constant reservoir release, R_t, in each period t, a line with slope R_t is placed so that it is tangent to the cumulative inflow curve. To the right of these points of tangency the release R_t exceeds the inflow Q_t. The maximum vertical distance between the cumulative inflow curve and the release line of slope R_t equals the maximum water deficit, and hence the required active storage capacity. Clearly, if the average of the releases R_t is greater than the mean inflow, a reservoir will not be able to meet the demand no matter what its active storage capacity.

An alternative way to identify the required reservoir storage capacity is to plot the cumulative non-negative deviations, $\sum_{\tau}^{t}(R_\tau - Q_\tau)$, and note the largest total deviation, as shown in Figure 11.13.

These graphical approaches do not account for losses. Furthermore, the method shown in Figure 11.12 is awkward if the desired releases in each period t are not the same. The equivalent method shown in Figure 11.13 is called the sequent peak method. If the sum of the desired releases does not exceed the sum of the inflows, calculations over at most two successive hydrological records of flows may be needed to identify the largest cumulative deficit inflow. After that the procedure will produce repetitive results. It is much easier to consider changing as well as constant release values when determining the maximum deficit by using the sequent peak method.

Sequent Peak Analyses

The sequent peak procedure is illustrated in Table 11.2. Let K_t be the maximum total storage requirement needed

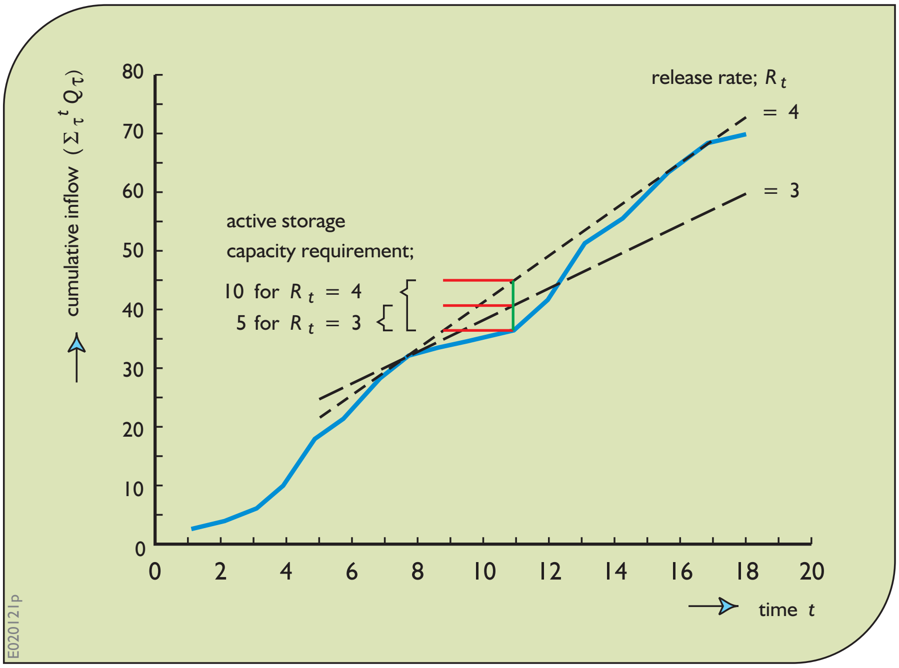

Figure 11.12. The Rippl or mass diagram method for identifying reservoir active storage capacity requirements.

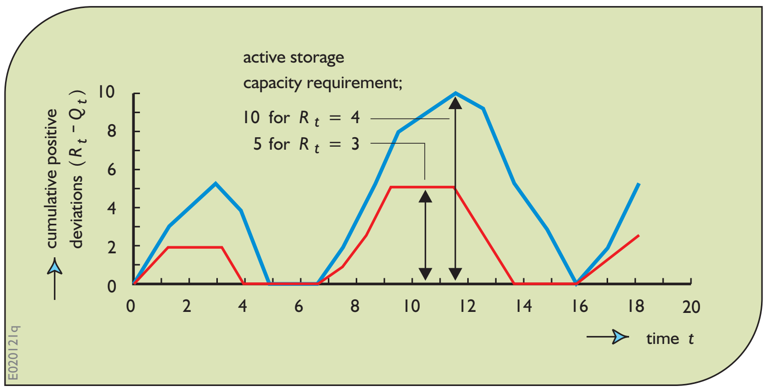

Figure 11.13. Alternative plot for identifying reservoir active storage capacity requirements.

for periods 1 up through period t. As before, let R_t be the required release in period t, and Q_t be the inflow in that period. Setting K_0 equal to 0, the procedure involves calculating K_t using Equation 11.30 (below) consecutively for up to twice the total length of record. This assumes that the record repeats itself to take care of the case when the critical sequence of flows occurs at the end of the streamflow record, as indeed it does in the example nine-period record of 1, 3, 3, 5, 8, 6, 7, 2 and 1.

$$\begin{aligned} K_t &= R_t - Q_t + K_{t-1} \quad \text{if positive,} \\ &= 0 \text{ otherwise} \end{aligned} \tag{11.30}$$

The maximum of all K_t is the required storage capacity for the specified releases R_t and inflows, Q_t. Table 11.2 illustrates this sequent peak procedure for computing the active capacity K_a, i.e. the maximum of all K_t, required to achieve a release $R_t = 3.5$ in each period given the series of nine streamflows. Note that this method does not require all releases to be the same.

2.2.2. Reservoir Storage–Yield Functions

Reservoir storage–yield functions define the minimum active storage capacity required to ensure a given constant release rate for a specified sequence of reservoir inflows. Mass diagrams, sequent peak analyses and linear optimization (Chapter 4) are three methods that can be used to define these functions. Given the same sequence of known inflows and specified releases, each method will provide identical results. Using optimization models,

time t	$(R_t - Q_t + K_{t-1})^+ = K_t$	
1	3.5 – 1.0 + 0.0 = 2.5	
2	3.5 – 3.0 + 2.5 = 3.0	
3	3.5 – 3.0 + 3.0 = 3.5	
4	3.5 – 5.0 + 3.5 = 2.0	
5	3.5 – 8.0 + 2.0 = 0.0	
6	3.5 – 6.0 + 0.0 = 0.0	
7	3.5 – 7.0 + 0.0 = 0.0	
8	3.5 – 2.0 + 0.0 = 1.5	
9	3.5 – 1.0 + 1.5 = 4.0	
1	3.5 – 1.0 + 4.0 = 6.5	
2	3.5 – 3.0 + 6.5 = 7.0	
3	3.5 – 3.0 + 7.0 = 7.5	K_a
4	3.5 – 5.0 + 7.5 = 6.0	
5	3.5 – 8.0 + 6.0 = 1.5	
6	3.5 – 6.0 + 1.5 = 0.0	repetition begins
7	3.5 – 7.0 + 0.0 = 0.0	
8	3.5 – 2.0 + 0.0 = 1.5	
9	3.5 – 1.0 + 1.5 = 4.0	

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Table 11.2. Illustration of the sequent peak procedure for computing active storage requirements.

it is possible to obtain such functions from multiple reservoirs and to account for losses based on storage volume surface areas, as will be discussed later.

There are two ways of defining a linear optimization (linear programming) model to estimate the active storage capacity requirements. One approach is to minimize the active storage capacity, K_a, subject to minimum required constant releases, Y, the yield. This minimum active storage capacity is the maximum storage volume, S_t, required, given the sequence of known inflows Q_t, and the specified yield, Y, in each period t. The problem is to find the storage volumes, S_t, and releases, R_t, that

$$\text{Minimize } K_a \tag{11.31}$$

subject to:

mass-balance constraints:

$$S_t + Q_t - R_t = S_{t+1} \quad t = 1, 2, \ldots, T; T + 1 = 1 \tag{11.32}$$

capacity constraints:

$$S_t \le K_a \quad t = 1, 2, \ldots, T \tag{11.33}$$

minimum release constraints:

$$R_t \ge Y \quad t = 1, 2, \ldots, T \tag{11.34}$$

for various values of the yield, Y.

Alternatively, one can maximize the constant release yield, Y,

$$\text{Maximize } Y \tag{11.35}$$

for various values of active storage capacity, K_a, subject to the same constraint Equations 11.32 through 11.34.

Constraints 11.32 and 11.34 can be combined to reduce the model size by T constraints.

$$S_t + Q_t - Y \ge S_{t+1} \quad t = 1, 2, \ldots, T; T + 1 = 1 \tag{11.36}$$

The solutions of these two linear programming models, using the nine-period flow sequence referred to above and solved for various values of yield or capacity, respectively, are plotted in Figure 11.14. The results are the same as could be found using the mass diagram or sequent peak methods.

There is a probability that the storage–yield function just defined will fail. A record of only nine flows, for example, is not very long and, hence, will not give one much confidence that they will define the critical low-flow period of the future. One rough way to estimate the reliability of a storage–yield function is to rearrange and rank the inflows in order of their magnitudes. If there are n ranked inflows, there will be $n+1$ intervals separating them. Assuming there is an equal probability that any future flow could occur in any interval between these ranked flows, there is a probability of $1/(n+1)$ that a future flow will be less than the lowest recorded flow. If that record low flow occurs during a critical low flow period, more storage may be required than indicated in the function.

Hence, for a record of only nine flows that are considered representative of the future, one can be only about 90% confident that the resulting storage–yield function will apply in the future. One can be only 90% sure of the predicted yield Y associated with any storage volume K. A much more confident estimate of the reliability of any derived storage–yield function can be obtained by using synthetic flows to supplement any measured streamflow record, taking parameter uncertainty into account (as discussed in Chapters 7, 8, and 9). This will provide alternative sequences as well as more intervals between ranked flows.

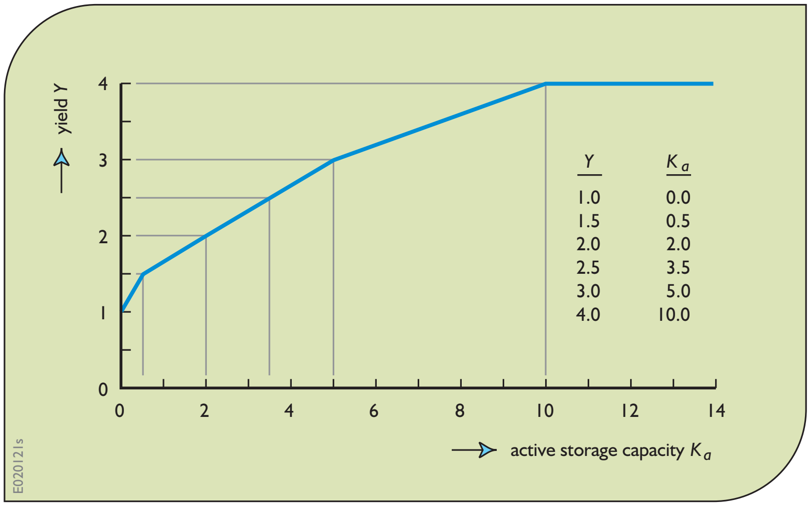

Figure 11.14. Storage–yield function for the sequence of flows 1, 3, 3, 5, 8, 6, 7, 2 and 1.

While the mass diagram and sequent peak procedures are relatively simple, they are not readily adaptable to reservoirs where evaporation losses and/or lake level regulation are important considerations, or to problems involving more than one reservoir. Mathematical programming (optimization) methods provide this capability. These optimization methods are based on mass-balance equations for routing flows through each reservoir. The mass-balance or continuity equations explicitly define storage volumes (and hence storage areas from which evaporation occurs) at the beginning of each period t.

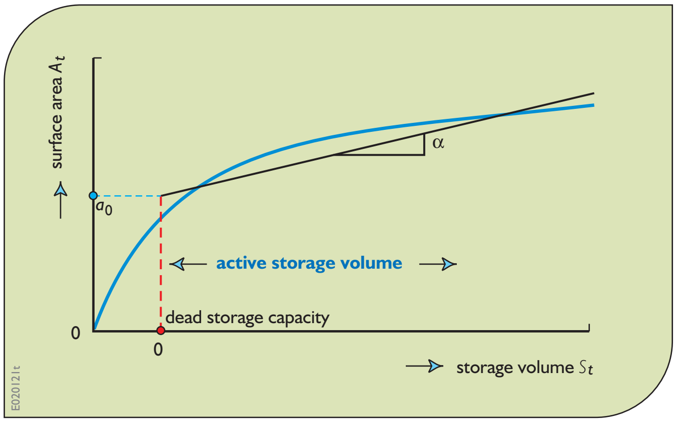

Figure 11.15. Storage surface area as a function of reservoir storage volume along with its linear approximation. The parameter *a* is the assumed increase in surface area associated with a unit increase in the storage volume.

2.2.3. Evaporation Losses

Evaporation losses, L_t, from lakes and reservoirs, if any, take place on their surface areas. Hence, to compute these losses, their surface areas must be estimated in each period t. Storage surface areas are functions of the storage volumes, S_t. These functions are typically concave, as shown in Figure 11.15.

In addition to the storage area–volume function, the seasonal surface water evaporation loss depths, $E_t^{\max}$, must be known. Multiplying the average surface area, A_t, based on the initial and final storage volumes, S_t and S_{t+1}, by the loss depth, $E_t^{\max}$, yields the volume of evaporation loss, L_t, in the period t. The linear approximation of that loss is

$$L_t = [a_0 + a(S_t + S_{t+1})/2]E_t^{\max} \tag{11.37}$$

Letting

$$a_t = 0.5aE_t^{\max} \tag{11.38}$$

the mass-balance equation for storage volumes that include evaporation losses in each period t can be approximated as:

$$(1 - a_t)S_t + Q_t - R_t - a_0E_t^{\max} = (1 + a_t)S_{t+1} \tag{11.39}$$

If Equation 11.39 is used in optimization models for identifying preliminary designs of a proposed reservoir, and if the active storage capacity turns out to be essentially zero, or just that required to provide for the fixed evaporation loss, $a_0E_t^{\max}$, then clearly any reservoir at the site is not justified. These mass-balance equations, together with any reservoir storage capacity constraints, should be removed from the model before resolving it again. This procedure is preferable to introducing 0,1 integer variables that will include the term $a_0 E_t^{\max}$ in Equation 11.39 only if the active storage volume is greater than 0 (using methods discussed in Chapter 4).

An alternative way to estimate evaporation loss that does not require a surface area–storage volume relationship, such as the one shown in Figure 11.15, is to define the storage elevation–storage volume function. Subtracting the evaporation loss depth from the initial surface elevation associated with the initial storage volume will result in an adjusted storage elevation, which in turn defines the initial storage volume after evaporation losses have been deducted. This adjusted initial volume can be used in continuity equations (Equation 11.32 or 11.36). This procedure assumes evaporation is only a function of the initial storage volume in each time period t. For relatively large volumes and short time periods, such an assumption is usually satisfactory.

2.2.4. Over and Within-Year Reservoir Storage and Yields

An alternative approach to modelling reservoirs is to separate out over-year storage and within-year storage, and to focus not on total reservoir releases, but on parts of the total releases that can be assigned specific reliabilities. These release components we call yields. To define these yields and the corresponding reservoir release rules, we divide this section into four parts. The first outlines a method for estimating the reliabilities of various constant annual minimum flows or yields. The second part discusses a modelling approach for estimating the over-year and within-year active storage capacities that are required to deliver specified annual and within-year yields, each having pre-specified reliabilities. The third and fourth parts expand this modelling approach to include multiple yields having different reliabilities, evaporation losses and the construction of reservoir operation rule curves using these flow release yields.

It will be convenient to illustrate the yield models and their solutions using a simple example of a single reservoir and two within-year periods per year. This example will be sufficient to illustrate the method that can be applied to larger systems having more within-year periods. Table 11.3 lists the nine years of available streamflow data for each within-year season at a potential reservoir site. These streamflows are used to compare the solutions of various yield models as well as to illustrate the concept of yield reliability.

time year y	within-year period Q_{ty} Q_{1y}	Q_{2y}	annual flow Q_y
1	1.0	3.0	4.0
2	0.5	2.5	3.0
3	1.0	2.0	3.0
4	0.5	1.5	2.0
5	0.5	0.5	1.0
6	0.5	2.5	3.0
7	1.0	5.0	6.0
8	2.5	5.5	8.0
9	1.5	4.5	6.0
total	**9.0**	**27.0**	**36.0**
average flow	1.0	3.0	4.0

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Table 11.3. Recorded unregulated historical streamflows at a reservoir site.

Reliability of Annual Yields

As discussed in Chapter 4, reservoirs can be operated so as to increase the dependable flow downstream. The flow that can always be depended upon at a particular site is commonly referred to as the 'firm yield' or 'safe yield'. These terms imply that this is what the reservoir will always be able to provide. Of course, this may not be true. If historical flows are used to determine this yield, then the resulting yield might be better called an 'historical yield'. Historical and firm yield are often used synonymously.

A flow yield is 100% reliable only if the sequence of flows in future years will never sum to a smaller amount than those that have occurred in the historic record. Usually one cannot guarantee this condition. Hence, associated with any historic yield is the uncertainty (a probability) that it might not always be available in the future. There are various ways of estimating this probability.

Referring to the nine-year streamflow record listed in Table 11.3, if no reservoir is built to increase the yields downstream of the reservoir site, the historic firm yield is the lowest flow on record, namely 1.0 that occurred in year 5. The reliability of this annual yield is the probability that the streamflow in any year will be greater than or equal to this value. In other words, it is the probability that this flow will be equalled or exceeded. The expected value of the exceedance probability of the lowest flow in an n-year record is approximately $n/(n+1)$, which for the $n = 9$ year flow record is $9/(9+1)$, or 0.90. This is based on the assumption that any future flow has an equal probability of being in any of the intervals formed by ordering the record of flows from the lowest to the highest value.

Now, rank the n flows of record from the highest to the lowest. Assign the rank m of 1 to the highest flow, and n to the lowest flow. In general, the expected probability p that any flow of rank m will be equalled or exceeded in any year is approximately $m/(n+1)$. An annual yield having a probability p of exceedance will be denoted as Y_p.

For independent events, the expected number of years until a flow of rank m is equalled or exceeded is the reciprocal of its probability of exceedance p, namely $1/p = (n+1)/m$. The recurrence time or expected time until a failure (a flow less than that of rank m) is the reciprocal of the probability of failure in any year. Thus, the expected recurrence time T_p associated with a flow having an expected probability p of exceedance is $1/(1-p)$.

2.2.5. Estimation of Active Reservoir Storage Capacities for Specified Yields

Over-Year Storage Capacity

A reservoir with active over-year storage capacity provides a means of increasing the magnitude and/or the reliabilities of various annual yields. For example, the sequent peak algorithm defined by Equation 11.30 provides a means of estimating the reservoir storage volume capacity required to meet various 'firm' yields, $Y_{0.9}$, associated with the nine annual flows presented in Table 11.3. The same yields can be obtained from a linear optimization model that minimizes active over-year storage capacity, K_a^o,

$$\text{Minimize } K_a^o \tag{11.40}$$

required to satisfy the following storage continuity and capacity constraint equations involving only annual storage volumes, S_y, inflows, Q_y, yields, Y_p, and excess releases, R_y. For each year y:

$$S_y + Q_y - Y_p - R_y = S_{y+1} \tag{11.41}$$

$$S_y \le K_a^o \tag{11.42}$$

Once again, if the year y is the last year of record, then year $y + 1$ is assumed to equal 1. For annual yields of 3 and 4, the over-year storage requirements are 3 and 8 respectively, as can be determined just by examining the right hand column of annual flows in Table 11.3.

The over-year model, Equations 11.40 through 11.42, identifies only annual or over-year storage requirements based on specified (known) annual flows, Q_y, and specified constant annual yields, Y_p. Within-year periods t requiring constant yields y_{pt} that sum to the annual yield Y_p may also be considered in the estimation of the required over-year and within-year or total active storage capacity, K_a. Any distribution of the over-year yield within the year that differs from the distribution of the within-year inflows may require additional active reservoir storage capacity. This additional capacity is called the within-year storage capacity.

The sequent peak method, Equation 11.30, can be used to obtain the total over-year and within-year active storage capacity requirements for specified within-year period yields, y_{pt}. Alternatively a linear programming model can be developed to obtain the same information along with associated reservoir storage volumes. The objective is to find the minimum total active storage capacity, K_a, subject to storage volume continuity and capacity constraints for every within-year period of every year. This model is defined as:

$$\text{minimize } K_a \tag{11.43}$$

subject to

$$S_{ty} + Q_{ty} - y_{pt} - R_{ty} = S_{t+1,\,y} \quad \forall t, y \tag{11.44}$$

$$S_{ty} \le K_a \quad \forall t, y \tag{11.45}$$

In Equation 11.44, if t is the final period T in year y, the next period $T + 1 = 1$ in year $y + 1$, or year 1 if y is the last year of record, n.

The difference in the active capacities resulting from these two models, Equations 11.40 through 11.42, and Equations 11.43 through 11.45, is the within-year storage requirement, K_a^w.

Table 11.4 shows some results from solving both of the above models. The over-year storage capacity requirements, K_a^0, are obtained from Equations 11.40 through 11.42. The combined over-year and within-year capacities, K_a, are obtained from solving Equations 11.43 through 11.45. The difference between the over-year storage capacity, K_a^0, required to meet only the annual yields and the total capacity, K_a, required to meet each specified within-year yield distribution of those annual yields is the within-year active storage capacity K_a^w.

Clearly, the number of continuity and reservoir capacity constraints in the combined over-year and within-year model (Equations 11.43–11.45) can become very large when the number of years n and within-year periods T are large. Each reservoir site in the river system will require $2nT$ continuity and capacity constraints. Not all these constraints are necessary, however. It is only a subset of the sequence of flows within the total record of flows that generally determines the required active storage capacity K_a of a reservoir. This is called the *critical period*. This critical period is often used in engineering studies to estimate the historical yield of any particular reservoir or system of reservoirs.

Even though the severity of future droughts is unknown, many planners accept the traditional practice of using the historical critical drought period for reservoir design and operation studies. They assume events that have happened in the past could happen again in the future. In some parts of the world, notably those countries in the lower portions of the southern hemisphere, historical records are continually proven to be unreliable indicators of future hydrological conditions. In these regions especially, synthetically generated flows based on statistical methods are more acceptable as a basis for yield estimation than seems to be the case elsewhere.

Over and Within-Year Storage Capacity

To begin the development of a smaller, but more approximate, model, consider each combined over-year and within-year storage reservoir to consist of two separate reservoirs in series (Figure 11.16). The upper reservoir is the over-year storage reservoir, whose capacity required for an annual yield is determined by an over-year model, such as Equations 11.40 through 11.42. The purpose of the 'downstream' within-year reservoir is to distribute as desired in each within-year period t the annual yield produced by the 'upstream' over-year reservoir. Within-year storage capacity would not be needed if the distribution

Table 11.4. Active storage requirements for various within-year yields.

annual yield $y^{0.9}$	within-year yields $t = 1$	$t = 2$	required active storage volume capacity: within-year K_a^w	over-year K_a^o	total K_a
3	0	3	1.0	3.0	4.0
	1	2	0.5	3.0	3.5
	2	1	1.5	3.0	4.5
	3	0	2.5	3.0	5.5
4	0	4	1.0	8.0	9.0
	1	3	0.0	8.0	8.0
	2	2	1.0	8.0	9.0
	3	1	2.0	8.0	10.0
	4	0	3.0	8.0	11.0

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of the average inflows into the upper over-year reservoir exactly coincided with the desired distribution of within-year yields. If they do not, within-year storage may be required. The two separate reservoir capacities summed together will be an approximation of the total active reservoir storage requirement needed to provide those desired within-year period yields.

Assume the annual yield produced and released by the over-year reservoir is distributed in each of the within-year periods in the same ratio as the average within-year inflows divided by the total average annual inflow. Let the ratio of the average period t inflow divided by the total annual inflow be β_t. The general within year model is to find the minimum within-year storage capacity, K_a^w, subject to within-year storage volume continuity and capacity constraints.

$$\text{Minimize } K_a^w \tag{11.46}$$

subject to

$$s_t + \beta_t Y_p - y_{pt} = s_{t+1} \quad \forall t \quad T+1 = 1 \tag{11.47}$$

$$s_t \leq K_a^w \quad \forall t \tag{11.48}$$

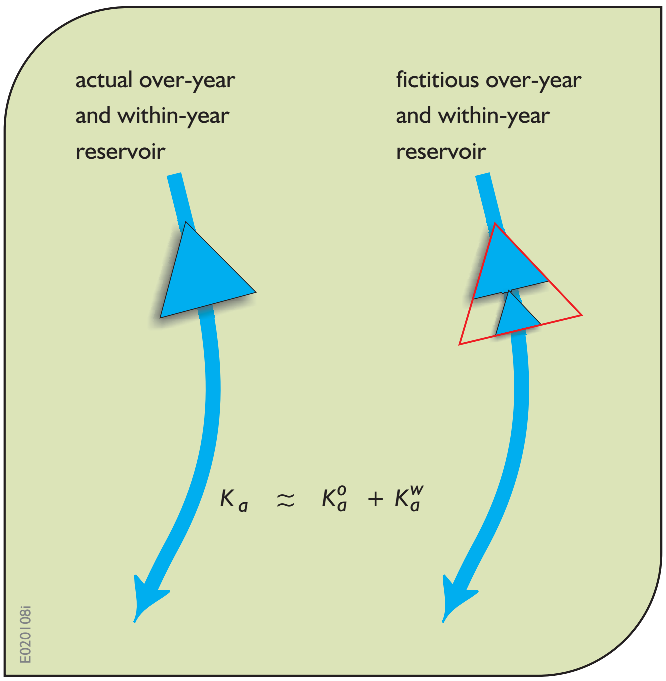

Figure 11.16. Approximating a combined over-year and within-year reservoir as two separate reservoirs, one for creating annual yields, the other for distributing them as desired in the within-year periods.

Since the sum of β_t over all within-year periods t is 1, the model guarantees that the sum of the unknown within-year yields, y_{pt}, equals the annual yield, Y_{pt}.

The over-year model, Equations 11.40 through 11.42, and within-year model, Equations 11.46 through 11.48, can be combined into a single model for an n-year sequence of flows:

$$\text{Minimize } K_a \tag{11.49}$$

subject to:

$$S_y + Q_y - Y_p - R_y = S_{y+1} \quad \forall y \quad \text{if } y = n, y+1 = 1 \tag{11.50}$$

$$S_y \leq K_a^o \quad \forall y \tag{11.51}$$

$$s_t + \beta_t Y_p - y_{pt} = s_{t+1} \quad \forall t \quad \text{if } t = T, T+1 = 1 \tag{11.52}$$

$$s_t \leq K_a^w \quad \forall t \tag{11.53}$$

$$\sum_t y_{pt} = Y_p \tag{11.54}$$

$$K_a \geq K_a^o + K_a^w \tag{11.55}$$

Constraint 11.54 is not required due to Equation 11.52, but is included here to make it clear that the sum of within-year yields will equal the over-year yield, and that such a constraint will be required for each yield of reliability p if multiple yields of different reliabilities are included in the model. In addition, constraint Equation 11.53 can be combined with Equation 11.55, saving a constraint. If this is done, the combined model contains $2n + 2T + 1$ constraints, compared to the more accurate model, Equations 11.43 through 11.45, that contains $2nT$ constraints.

If the fractions β_t are based on the ratios of the average within-year inflow divided by average annual inflow in the two within-year periods shown in Table 11.3, then 0.25 of the total annual yield flows into the fictitious within-year reservoir in period $t = 1$, and 0.75 of the total annual yield flows into the reservoir in period $t = 2$. Suppose the two desired within-year yields are to be 3 and 0 for periods 1 and 2 respectively. The total annual yield, $Y_{0.9}$, is 3. Assuming the natural distribution of this annual yield of 3 in period 1 is 0.25 $Y_{0.9} = 0.75$, and in period 2 it is 0.75 $Y_{0.9} = 2.25$, the within-year storage required to redistribute these yields of 0.75 and 2.25 to become 3 and 0, respectively, is $K_a^w = 2.25$. From Table 11.4, we can see an annual yield of 3 requires an over-year storage capacity of 3. Thus, the estimated total storage capacity required to provide yields of 3 and 0 in periods 1 and 2 is the over-year capacity of 3 plus the within-year capacity of 2.25 equalling 5.25. This compares with 3 plus 2.5 of actual within-year capacity required for a total of 5.50, as indicated in Table 11.4.

There are ways to reduce the number of over-year constraints without changing the solution of the over-year model. Sequences of years whose annual inflow values equal or exceed the desired annual yield can be combined into one constraint. If the yield is an unknown variable, then the mean annual inflow can be used since it is the upper limit of the annual yield. For example in Table 11.3 note that the last three years and the first year have flows equal or greater than 4, the mean annual inflow. Thus,these four successive years can be combined into a single continuity equation:

$$S_7 + Q_7 + Q_8 + Q_9 + Q_1 - 4Y_p - R_7 = S_2 \tag{11.56}$$

This saves a total of three over-year continuity constraints and three over-year capacity constraints. Note that the excess release, R_7, represents the excess release in all four periods. Furthermore, not all reservoir capacity constraint Equation 11.51 are needed, since the initial storage volumes in the years following low flows will probably be less than the over-year capacity.

There are many ways to modify and extend this yield model to include other objectives, fixed ratios of the unknown annual yield for each within-year period, and even multiple yields having different exceedance probabilities p.

The number of over-year periods being modelled compared to the number of years of flow records determines the highest exceedance probability or reliability a yield can have, e.g. 9/10 or 0.9 in the nine-year example used here. If yields having lower reliabilities are desired, such as a yield with a reliability of 0.80, then the yield variable Y_P ($Y_{0.90}$) can be omitted from Equation 11.50 in the critical year that determines the required over-year capacity for a 0.90 reliable yield. (Since some outflow might be expected, even if it is less than the 90% reliable yield, the outflow could be forced to equal the inflow for that year.) If a 0.70 reliable yield is desired, then the yield variables in the two most critical years can be omitted from Equation 11.50, and so on.

The number of years of yield failure determines the estimated reliability of each yield. An annual yield that fails in f out of n years has an estimated probability $(n-f)/(n+1)$ of being equalled or exceeded in any future year. Once the desired reliability of a yield is known, the modeller must select the appropriate failure years and specify the permissible extent of failure in those f failure years.

To consider different yield reliabilities p, let the parameter α_y^p be a specified value between 0 and 1 that indicates the extent of a failure in year y associated with an annual yield having a reliability of p. When α_y^p is 1 there is no failure, and when it is less than 1 there is a failure, but a proportion of the yield Y_p equal to α_y^p is released. Its value is in part dependent on the consequences of failure and on the ability to forecast when a failure may occur and to adjust the reservoir operating policy accordingly.

The over-year storage continuity constraints for n years can now be written in a form appropriate for identifying any single annual yield Y_p having an exceedance probability p:

$$S_y + Q_y - \alpha_y^p Y_p - R_y = S_{y+1} \quad \forall y$$
$$\text{if } y = n, y+1 = 1 \tag{11.57}$$

When writing Equation 11.57, the failure year or years should be selected from among those in which permitting a failure decreases the required reservoir capacity K_a. If a failure year is selected that has an excess release, no reduction in the required active storage capacity will result and the reliability of the yield may be higher than intended.

The critical year or years that determine the required active storage volume capacity may be dependent on the yield itself. Consider, for example, the seven-year sequence of annual flows (4, 3, 3, 2, 8, 1, 7) whose mean is 4. If a yield of 2 is desired in each of the seven years, the critical year requiring reservoir capacity is year 6. If a yield of 4 is desired (again assuming no losses), the critical years are years 2 through 4. The streamflows and yields in these critical years determine the required over-year storage capacity. The failure years, if any, must be selected from within the critical low flow periods for the desired yield.

When the magnitudes of the yields are unknown, some trial-and-error solutions may be necessary to ensure that any failure years are within the critical period of years for the associated yields. To ensure a wider range of applicable yield magnitudes, the year having the lowest flow within the critical period should be selected as the failure year if only one failure year is selected. Even though the actual failure year may follow that year, the resulting required reservoir storage volume capacity will be the same.

Multiple Yields and Evaporation Losses

The yield models developed so far define only single annual and within-year yields. Incremental secondary yields having lower reliabilities can also be included in the model. Referring to the nine-year streamflow record in Table 11.3, assume that two yields are desired, one 90% reliable and the other 70% reliable. Let $Y_{0.9}$ and $Y_{0.7}$ represent those annual yields having reliabilities of 0.9 and 0.7, respectively. The incremental secondary yield $Y_{0.7}$ represents the amount in addition to $Y_{0.9}$ that is only 70% reliable. Assume that the problem is one of estimating the appropriate values of $Y_{0.9}$ and $Y_{0.7}$, their respective within-year allocations y_{pt}, and the total active reservoir capacity K_a that maximizes some function of these yield and capacity variables.

In this case, the over-year and within-year continuity constraints can be written

$$S_y + Q_y - Y_{0.9} - \alpha_y^{0.7} Y_{0.7} - R_y = S_{y+1} \quad \forall y$$
$$\text{if } y = n,\ y+1 = 1 \qquad (11.58)$$

$$s_t + \beta_t (Y_{0.9} + Y_{0.7}) - y_{0.9,t} - y_{0.7,t} = s_{t+1} \quad \forall t$$
$$\text{if } t = T,\ T+1 = 1 \qquad (11.59)$$

Now an additional constraint is needed to ensure that all within-year yields of a reliability p sum to the annual yield of the same reliability. Selecting the 90% reliable yield,

$$\sum_t y_{0.9,t} = Y_{0.9} \qquad (11.60)$$

Evaporation losses must be based on an expected storage volume in each period and year, since the actual storage volumes are not identified using these yield models. The approximate storage volume in any period t in year y can be defined as the initial over-year volume S_y, plus the estimated average within-year volume $(s_t + s_{t+1})/2$. Substituting this storage volume into Equation 11.37 (see also Figure 11.15) results in an estimated evaporation loss L_{yt}:

$$L_{yt} = [a_0 + a(S_y + (s_t + s_{t+1})/2)]E_t^{\max} \qquad (11.61)$$

Summing L_{yt} over all within-year periods t defines the estimated annual evaporation loss, E_y:

$$E_y = \sum_t [a_0 + a(S_y + (s_t + s_{t+1})/2)]E_t^{\max} \qquad (11.62)$$

This annual evaporation loss applies, of course, only when there is a non-zero active storage capacity requirement. These annual evaporation losses can be included in the over-year continuity constraints, such as Equation 11.58. If they are, the assumption is being made that their within-year distribution will be defined by the fractions β_t. This may not be realistic. If it is not, an alternative would be to include the average within-year period losses, L_t, in the within-year constraints.

The average within-year period loss, L_t, can be defined as the sum of each loss L_{yt} defined by Equation 11.61 over all years y divided by the total number of years, n:

$$L_t = \sum_y^n [a_0 + a(S_y + (s_t + s_{t+1})/2)]E_t^{\max}/n \qquad (11.63)$$

This average within-year period loss, L_t, can be added to the within-year's highest reliability yield, y_{pt}, forcing greater total annual yields of all reliabilities to meet corresponding total within-year yield values. Hence, combining Equations 11.60 and 11.61, for p equal to 0.9 in the example,

$$Y_p = \sum_t \left\{ y_{pt} + \sum_y^n [a_0 + a(S_y + (s_t + s_{t+1})/2)]E_t^{\max}/n \right\} \qquad (11.64)$$

Since actual reservoir storage volumes in each period t of each year y are not identified in this model, system performance measures that are functions of those storage volumes, such as hydroelectric energy or reservoir recreation, are only approximate. Thus, as with any of these screening models, any set of solutions should be evaluated and further improved using more precise simulation methods.

Simulation methods require reservoir operating rules. The information provided by the solution of the yield model can help in defining a reservoir operating policy for such simulation studies.

Reservoir Operation Rules

Reservoir operation rules, as discussed in Chapters 4, 7 and 8, are guides for those responsible for reservoir operation. They apply to reservoirs being operated in a steady-state condition, i.e. not filling up immediately after construction or being operated to meet a set of new and temporary objectives. There are several types of rules, but each indicates the desired or required reservoir release or storage volume at any particular time of year. Some rules identify storage volume targets (rule curves) that the operator is to maintain, if possible, and others identify storage zones, each associated with a particular release policy. This latter type of rule can be developed from the solution of the yield model.

To construct an operation rule that identifies storage zones, each having a specific release policy, the values of the dead and flood storage capacities, K_D and K_f, are needed together with the over-year storage capacity, K_a^o, and within-year storage volumes, s_t, in each period t. Since both K_a^o and all s_t derived from the yield model are for all yields, Y_p, being considered, it is necessary to determine the over-year capacities and within-year storage volumes required to provide each separate within-year yield, y_{pt}. Plotting the curves defined by the respective over-year capacity plus the within-year storage volume ($K_a^o + s_t$) in each within-year period t will define a zone of storage whose yield releases y_{pt} from that zone should have a reliability of at least p.

For example, assume again a nine-year flow record and ten within-year periods. Of interest are the within-year yields, $y_{0.9,t}$ and $y_{0.7,t}$, having reliabilities of 0.9 and 0.7. The first step is to compute the over-year storage capacity requirement, K_a^o, and the within-year storage volumes, s_t, for just the yields $y_{0.9,t}$. The sum of these values, $K_a^o + s_t$, in each period t can be plotted as illustrated in Figure 11.17.

The sum of the over-year capacity and within-year volume $K_a^o + s_t$ in each period t defines the zone of active storage volumes for each period t required to supply the within-year yields $y_{0.9,t}$. If the storage volume is in the shaded zone shown in Figure 11.17, only the yields $y_{0.9,t}$ should be released. The reliability of these yields, when simulated, should be about 0.9. If at any time t the actual reservoir storage volume is within this zone, then reservoir releases should not exceed those required to meet the yield $y_{0.9,t}$ if the reliability of this yield is to be maintained.

The next step is to solve the yield model for both yields $Y_{0.9}$ and $Y_{0.7}$. The resulting sum of over-year storage capacity and within-year storage volumes can be plotted over the first zone, as shown in Figure 11.18.

If at any time t the actual storage volume is in the second, lighter-shaded, zone in Figure 11.18, the release should be the sum of both the most reliable yield, $y_{0.9,t}$ and the incremental secondary yield $y_{0.7,t}$. If only these releases are made, then the probability of being in that zone, when simulated, should be about 0.7. If the actual storage volume is greater than the total required over-year storage capacity K_a^o plus the within-year volume s_t, the non-shaded zone in Figure 11.18, then a release can

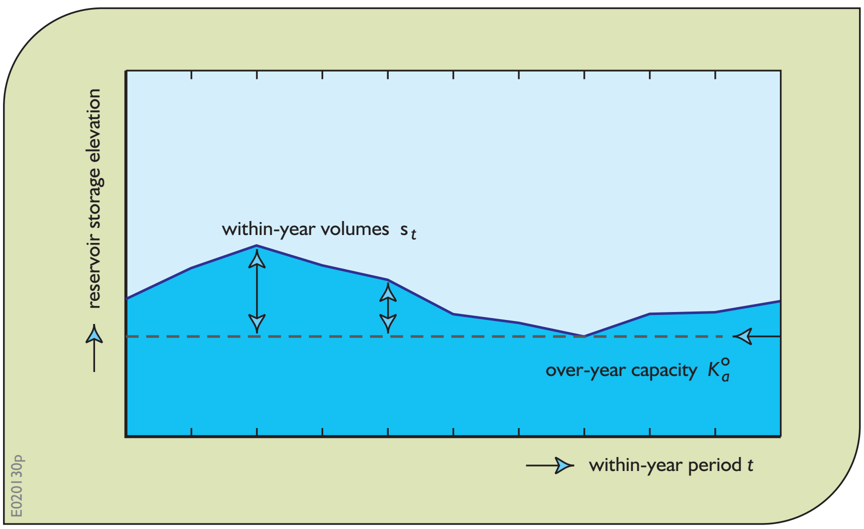

Figure 11.17. Reservoir release rule showing the identification of the most reliable release zone associated with the within-year yields $y_{0.9,t}$.

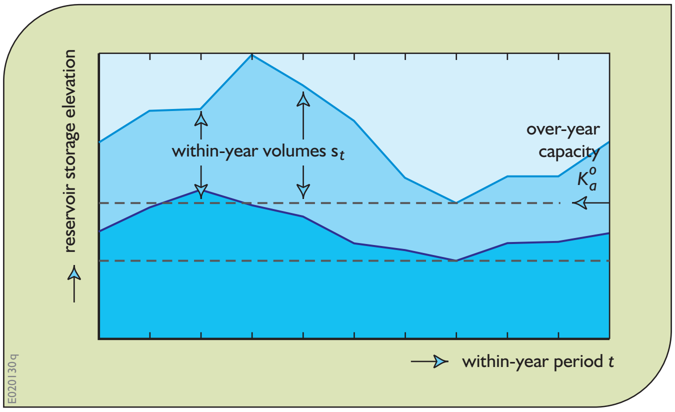

Figure 11.18. Reservoir release rule showing the identification of the second most reliable release zone associated with the total within-year yields $y_{0.9,t} + y_{0.7,t}$.

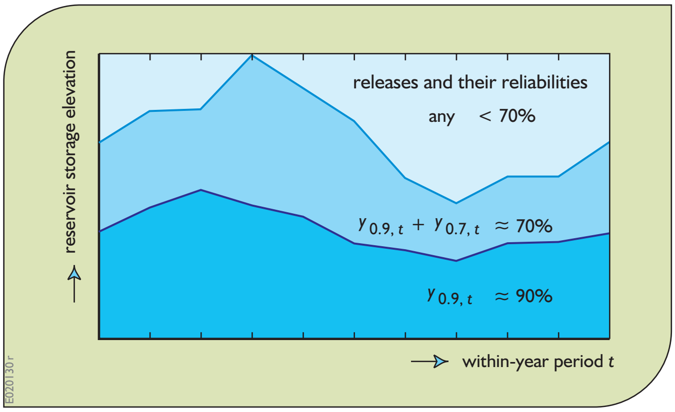

Figure 11.19. Reservoir release rule defined by the yield model.

be made to satisfy any down-stream demand. Converting storage volume to elevation, this release policy is summarized in Figure 11.19.

These yield models focus only on the active storage capacity requirements. They can be a part of a model that includes flood storage requirements as well (as discussed later in this chapter). If the actual storage volume is within the flood control zone in the flood season, releases should be made to reduce the actual storage to a volume no greater than the total capacity less the flood storage capacity.

Once again, reservoir rules developed from simplified models such as this yield model are only guides, and once developed they should be simulated, evaluated and refined prior to their actual adoption.

2.3. Wetlands and Swamps

Wetlands and swamps are important elements of the water resources system. They provide regulating functions with respect to both quantity (e.g. flood attenuation) and quality (e.g. self-purification). Most wetlands support valuable ecological systems. Appendix A (Section 4) describes the natural processes involved in wetlands.

From a quantitative point of view, wetlands are comparable with (shallow) lakes. Lakes are usually located on streams or rivers, while most wetlands are separate elements connected to a stream or river only in wet periods, either by flooding from the stream or river or by draining to it. Some wetlands are part of river systems. Examples are the Sudd Swamps in the Nile River, the Pantanal in the Paraguay River in South America and the marshlands in southern Iraq at the confluence of the Euphrates and Tigris Rivers. In these cases wetlands can be modelled as lakes or reservoirs, as described in the previous section. If they are separate from the river system, wetlands can be included as indicated in Figure 11.20. The lateral flow $Q_3(t)$ can be bi-directional.

2.4. Water Quality and Ecology

River basin models focusing on water quantities are mostly used to investigate whether sufficient water is available to satisfy the various use functions (off-stream and in-stream), and to identify measures to match supply and demand. The core of most river basin models consists of keeping track of the water balance of the whole river basin. The analysis of water quality and ecology is mostly done 'off-line', using another model for a specific part of the system, e.g. a river stretch, reservoir or groundwater system. These models will be described in Chapter 12.

Such an off-line approach makes sense. There is little feedback from water quality to quantity (except in cases where minimum flows are required to maintain a minimum water quality level). Using separate water quality models for parts of the system makes it possible to include more temporal and spatial detail and to include more complex water quality processes. Environmental flow requirements can be included in river basin models by defining specific flow regime demands (quantity, velocity, dynamics and the like) at certain locations in the river basin.

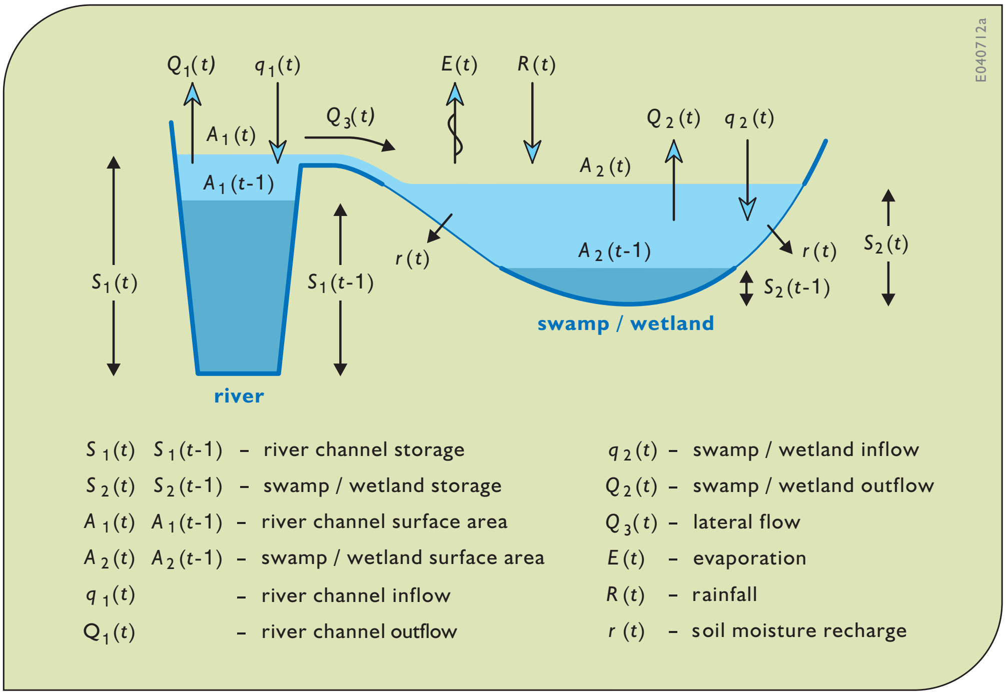

Figure 11.20. Schematization of the link between wetlands and a river system.

3. Modelling the Socio-Economic Functions In a River Basin

3.1. Withdrawals and Diversions

Major demands for the withdrawal of water include those for domestic or municipal uses, industrial uses (including cooling water) and agricultural uses (including irrigation). These uses generally require the withdrawal of water from a river system or other surface or groundwater body. The water withdrawn may be only partially consumed, and that which is not consumed may be returned to the river system, perhaps at a different site, at a later time period, and containing different concentrations of constituents.

Water can also be allocated to instream uses that alter the distribution of flows in time and space. Such uses include:

- reservoir storage, possibly for recreational use as well as for water supply
- flow augmentation, possibly for water quality control, navigation or ecological benefits
- hydroelectric power production.

The instream uses may complement or compete with each other or with various off-stream municipal, industrial and agricultural demands. One purpose of developing management models of river basin systems is to help derive policies that will best serve these multiple uses, or at least identify the tradeoffs among the multiple purposes and objectives.

The allocated flow q_t^s to a particular use at site s in period t must be no greater than the total flow available, Q_t^s, at that site and in that period:

$$q_t^s \leq Q_t^s \tag{11.65}$$

The quantity of water that any particular user expects to receive in each particular period is termed the *target allocation*. Given an annual (known or unknown) target allocation T^s at site s, some (usually known) fraction, f_t^s, of that annual target allocation will be expected in each within-year period t. If the actual allocation, q_t^s, is less than the target allocation, $f_t^s T^s$, there will be a deficit, D_t^s. If the allocation is greater than the target allocation, there will be an excess, E_t^s. Hence, to define those unknown variables, the following constraint equation can be written for each applicable period t:

$$q_t^s = f_t^s T^s - D_t^s + E_t^s \tag{11.66}$$

Even though allowed, one would not expect a solution to contain non-zero values for both D_t^s and E_t^s.

Whether or not any deficit or excess allocation should be allowed at any demand site s depends on the quantity of water available and the losses or penalties associated with deficit or excess allocations to that site. At sites where the benefits derived in each period are independent of the allocations in other periods, the losses associated with deficits and the losses or benefits associated with excesses can be defined in each period t (Chapter 10). For example, the benefits derived from the allocation of water for hydropower production in period t will in some cases be essentially independent of previous allocations.

For any use in which the benefits are dependent on a sequence of allocations, such as at irrigation sites, the benefits may be based on the annual (or growing season) target water allocations T^s and their within season distributions, $f_t^s T^s$. In these cases one can define the benefits from those water uses as functions of the unknown season or annual targets, T^s, where the allocated flows q_t^s must be no less than the specified fraction of that unknown target;

$$q_t^s \geq f_t^s T^s \quad \text{for all relevant } t \tag{11.67}$$

If, for any reason, an allocation q_t^s must be zero, then clearly from Equation 11.67 the annual or growing season target allocation T^s would be zero, and presumably so would be the benefits associated with that target value.

Water stored in reservoirs can often be used to augment downstream flows for instream uses such as recreation, navigation and water quality control. During natural low-flow periods in the dry season, it is not only the increased volume but also the lower temperature of the augmented flows that may provide the only means of maintaining certain species of fish and other aquatic life. Dilution of wastewater or runoff from non-point sources may be another potential benefit from flow augmentation. These and other factors related to water quality management are discussed in greater detail in Chapter 12.

The benefits derived from navigation on a potentially navigable portion of a river system can usually be expressed as a function of the stage or depth of water in various periods. Assuming known stream or river flow-stage relationships at various sites in the river, a possible constraint might require at least a minimum acceptable depth, and hence flow, for those sites.

3.2. Domestic, Municipal and Industrial Water Demand

Domestic, municipal and industrial (DMI) water demands are typically based on projections of population and socio-economic activities. The domestic demand is the household demand. The municipal demand covers the water requirements of the commercial sector (shops, department stores, hotels and so on) and for the services sector (hospitals, offices, schools and others). In most river basins, the DMI demand will be small compared to the demand for irrigation. However, given the importance of this demand for human health and economic developments, it is often given first priority. The preferred source for drinking water (domestic and municipal) is often groundwater which requires only limited treatment. When sufficient groundwater of good quality is not available, surface water will be used, in most cases in combinati4on with advanced treatment facilities.

About 80% to 90% of the water abstracted for DMI will usually be returned to the system as wastewater. About 95% of cooling water is also typically returned to the basin. In river basin studies, the total demand has to be taken into account, as this amount is actually withdrawn from the system. The water not used will be returned to the system, possibly at another location, in a future time period and containing more pollutants because of its use.

Predicting DMI demands can be difficult. A common approach is to use the demand predictions of the public water supply authorities in the basin. They base their predictions on estimates of population growth, growth in demand per capita and so on, often using statistical trends. As these authorities have a good knowledge of what is going on in their area, their estimates are often the most reasonable ones to use, although they often tend to overestimate the demand.

In many cases these predictions are not available or not sufficiently reliable, and it is then usually necessary to collect basic information on population and socio-economic activities and expected growth rates. Domestic use can be estimated on the basis of water consumption per capita and coverage rates, taking into account differences in social strata. Municipal demand is often assumed to be a percentage of the domestic water demand (usually in the range of 15–35%). Industrial water demand is the most difficult to estimate, as it depends on the type of industry and the production processes being used. Not only is it difficult to get all the data needed to make accurate predictions, but future economic and technological developments are very uncertain. In most cases estimates are made on the basis of statistical projections or of water demand per industrial employee (for which data are easier to get), depending on the type of industry.

Once a total demand for DMI has been determined, projections have to be made for the losses in the system. 'Unaccounted for' water losses depend on the condition and maintenance of the distribution system and should be taken into account. In some systems such losses can exceed 40%.

3.3. Agricultural Water Demand

Most agricultural areas in the world depend on rain as the primary source of water. During dry periods, additional water may be supplied through irrigation systems. In semi-arid and arid countries, irrigation is an absolute requirement to enable agricultural production. In those cases, agricultural water is often the dominant demand category. The irrigation demand in Egypt accounts for 90% of the total net water demand, and in the western United States for about 75% of the total.

Irrigation water demand depends on a large number of factors, which can be grouped under four headings:

- hydro-meteorological conditions
- types of crops
- soil types
- irrigation practices and water use efficiency.

Crops transpire as they grow. At the same time, there is evaporation from the soil surface. The combined quantity of water used under conditions of optimum availability is known as 'consumptive use' or 'evapotranspiration' (ET). The potential evapotranspiration, E_p, is estimated from the following equation:

$$E_p = c_p ET_0 \tag{11.68}$$

in which:

E_p = potential crop evapotranspiration (mm per time unit)

c_p = crop coefficient

ET_0 = reference crop evapotranspiration (mm per time unit)

The reference crop evapotranspiration, ET_0, can be computed using a standard FAO method (FAO, 1998) based on sunshine, temperature, humidity and wind speed. The crop coefficient, c_p, is related to the type of crop and the growth stage.

Based on crop water demand (E_p), the crop irrigation demand ($C_d = E_p -$ effective rainfall), field irrigation demand ($F_d = C_d +$ field losses) and project irrigation demand ($P_d = F_d +$ distribution losses) can be calculated.

The calculation of irrigation demand is complex as farmers use many different cropping patterns and cropping calendars. Several computer programs (e.g. the FAO's CROPWAT) are available to support that calculation, and many river basin simulation packages (see Section 4.3) have modules that estimate the irrigation demand.

3.4. Hydroelectric Power Production

Hydropower plants, Figure 11.21, convert the energy from the flowing water into mechanical and then electrical energy. These plants, containing turbines and generators, are installed either in or adjacent to dams. Pipelines (penstocks) carry water under pressure from the reservoir to the powerhouse. Power transmission systems transport energy from the powerhouse to where it is needed.

The principal advantages of hydropower are the absence of polluting emissions during operation, its capability to respond quickly to changing utility load demands, and its relatively low operating costs. Disadvantages can include high initial capital cost, potential site-specific impacts and cumulative environmental ones. The potential environmental impacts of hydropower projects include altered flow regimes below storage reservoirs, water quality degradation, mortality of fish that pass through hydroelectric turbines, blockage of fish migration and flooding of terrestrial ecosystems by impoundments. Proper design and operation of hydropower projects can mitigate some of these impacts (see also WCD, 2000). Hydroelectric projects can also provide beneficial effects such as recreation in reservoirs or in tailwaters below dams.

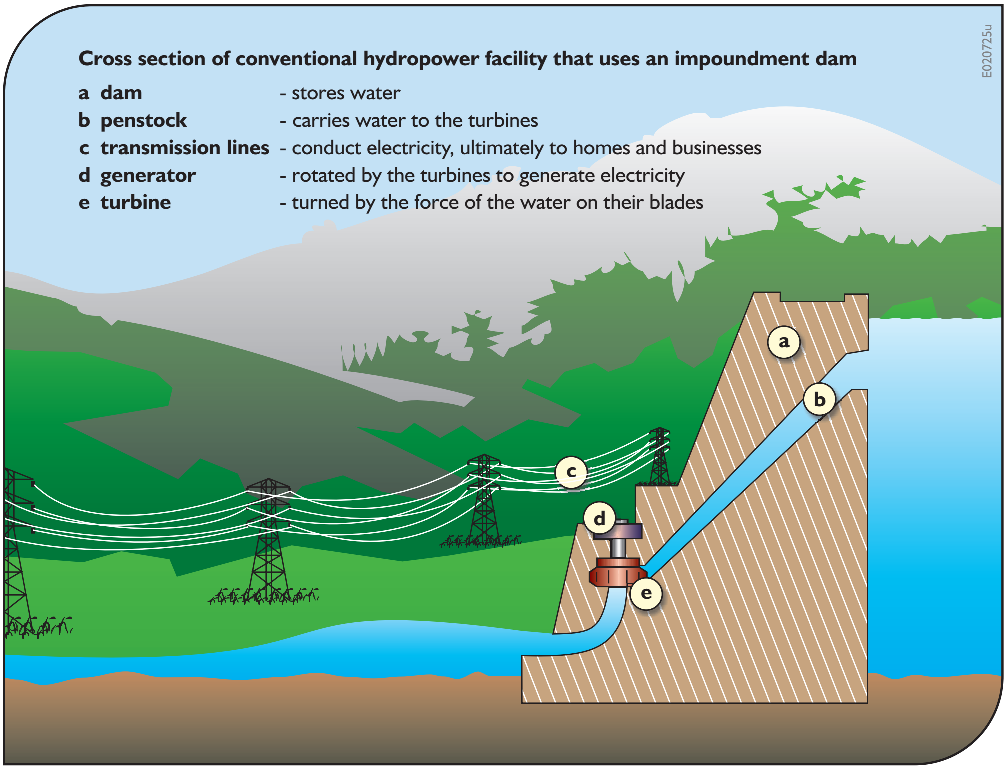

Figure 11.21. Hydropower plant and system components.

Hydropower technology can be grouped into two types: *conventional* and *pumped storage*. Conventional plants use the available water from a river, stream, canal system or reservoir to produce electrical energy. In conventional multipurpose reservoirs and run-of-river systems, hydropower production is just one of many competing purposes for which water resources may be used. Competing water uses can include irrigation, flood control, navigation, and municipal and industrial water supply. Pumped storage plants pump the water, usually through a reversible turbine, from a lower to an upper reservoir. While pumped storage facilities are net energy consumers, they are income producers. They are valued by the utilities because they can be rapidly brought online to operate in a peak power production mode when energy demand and prices are highest. The pumping to replenish the upper reservoir is performed during off-peak hours when electricity costs are lowest. This process benefits the utility by increasing the load factor and reducing the cycling of its base-load units. In most cases, pumped storage plants run a full cycle every twenty-four hours (USDOE, 2002).

Run-of-river plants use the natural flow of the river and produce relatively little change in the stream channel and streamflow, in contrast to a peaking plant that impounds water when the energy is needed. A storage project extensively impounds and stores water during high-flow periods to augment the water available during low-flow periods, allowing the flow releases and power production to be more constant. Many projects combine the modes.

The power capacity of a hydropower plant is primarily the function of the hydraulic head and the flow rate through the turbines. The hydraulic head is the elevation difference the water falls in passing through the plant or to the tailwater, whichever elevation difference is less. Project design may concentrate on either of these variables or both, and on the hydropower plant's installed capacity.

The production of hydroelectric energy during any period at any particular reservoir site is dependent on the installed plant capacity; the flow through the turbines; the average effective productive storage head; the number of hours in the period; the plant factor (the fraction of time that energy is produced); and a constant for converting the product of flow, head and plant efficiency to electrical energy. The kilowatt-hours of energy, KWH_t, produced in period t is proportional to the product of the plant efficiency, e, the productive storage head, H_t, and the flow, q_t, through the turbines.

A cubic metre of water, weighing 10^3 kg, falling a distance of one metre, acquires 9.81×10^3 joules (Newton-metres) of kinetic energy. The energy generated in one second equals the watts (joules per second) of power produced. Hence, an average flow of q_t cubic metres per second falling a height of H_t *metres* in period t yields $9.81 \times 10^3 q_t H_t$ watts or $9.81\ q_t H_t$ kilowatts of power. Multiplying by the number of hours in period t yields the kilowatt-hours of energy produced from an average flow rate of q_t. The total kilowatt-hours of energy, KWH_t, produced in period t, assuming 100% efficiency in conversion of potential to electrical energy, is

$$
\begin{aligned}
KWH_t &= 9.81 q_t H_t(\text{seconds in period } t)/ \\
&\qquad (\text{seconds per hour}) \\
&= 9.81 q_t H_t(\text{seconds in period } t)/3600 \qquad (11.69)
\end{aligned}
$$

Since the total flow, Q_t^T through the turbines in period t equals the average flow rate q_t times the number of seconds in the period, the total kilowatt-hours of energy produced in period t, given a plant efficiency of e, equals

$$
\begin{aligned}
KWH_t &= 9.81 Q_t^T H_t e/3600 \\
&= 0.002725 Q_t^T H_t e \qquad (11.70)
\end{aligned}
$$

The energy required for pumped storage, where instead of producing energy the turbines are used to pump water up to a higher level, is

$$KWH_t = 0.002725 Q_t^T H_t / e \qquad (11.71)$$

For Equations 11.70 and 11.71, Q_t^T is in cubic metres and H_t is in metres. The storage head, H_t, is the vertical distance between the water surface elevation in the lake or reservoir that is the source of the flow through the turbines and the maximum of either the turbine elevation or the downstream discharge elevation. In variable head reservoirs, storage heads are functions of storage volumes. In optimization models for capacity planning these heads and the turbine flows are among the unknown variables. The energy produced is proportional to the product of these two unknown variables. This results in non-separable functions in equations that must be written at each hydroelectric site for each time period t.

A number of ways have been developed to convert these non-separable energy production functions to separable ones for use in linear optimization models for estimating design and operating policy variable values. These methods inevitably increase the model complexity and the number of its variables and constraints. For a preliminary screening of hydropower capacities prior to a more detailed analysis (e.g. using simulation or other non-linear or discrete dynamic programming methods) one can:

1. Solve the model using both optimistic and pessimistic assumed fixed head values.
2. Compare the actual derived heads with the assumed ones and adjust the assumed heads.
3. Resolve the model.
4. Compare the capacity values.

From this iterative process one should be able to identify the range of hydropower capacities that can then be further refined using simulation.

Alternatively, assumed average heads, H_t^o, and flows, Q_t^o, can be used in a linear approximation of the non-separable product terms, $Q_t^T H_t$:

$$Q_t^T H_t = H_t^o Q_t^T + Q_t^o H_t - Q_t^o H_t^o \qquad (11.72)$$

Again, the model may need to be solved several times in order to identify reasonably accurate average flow and head estimates in each period t.

The amount of electrical energy produced is limited by the installed kilowatts of plant capacity, P, as well as on the plant factor, p_t. The plant factor is a measure of hydroelectric power plant use. Its value depends on the characteristics of the power system and the demand pattern for hydroelectric energy. The plant factor is defined as the average power load on the plant for the period divided by the installed plant capacity. The plant factor accounts for the variability in the demand for hydropower during each period t. This factor is usually specified by those responsible for energy production and distribution. It may or may not vary for different time periods.

The total energy produced cannot exceed the product of the plant factor, p_t, the number of hours, h_t, in the period, and the plant capacity, P, measured in kilowatts:

$$KWH_t \leq P h_t p_t \qquad (11.73)$$

3.5. Flood Risk Reduction

Two types of structural alternatives exist for flood risk reduction. One is flood storage capacity in reservoirs designed to reduce downstream peak flood flows. The other is channel enhancement and/or flood-proofing works that will contain peak flood flows and reduce damage. This section introduces methods of modelling both of these alternatives for inclusion in either benefit–cost or cost-effectiveness analyses. The latter analyses apply to situations in which a significant portion of the flood control benefits cannot be expressed in monetary terms and the aim is to provide a specified level of flood protection at minimum cost.

The discussion will first focus on the estimation of flood storage capacity in a single reservoir upstream of a potential flood damage site. This analysis will then be expanded to include downstream channel capacity improvements. Each of the modelling methods discussed will be appropriate for inclusion in multipurpose river basin planning (optimization) models having longer time step durations than those required to predict flood peak flows. Additional detail can be found in Appendix D.

3.5.1. Reservoir Flood Storage Capacity

A common approach for estimating reservoir flood storage capacity is based on expected damage reduction associated with various storage capacities in a particular reservoir. A relationship between flood storage capacity and expected flood damage reduction can be obtained. This functional relationship can then be used within a multipurpose reservoir model that considers other uses of the reservoir and its stored water. This approach avoids having to include short time steps applicable to most floods in an overall multipurpose river basin model.

Consider a reservoir upstream of a potential flood damage site along a river. The question is how much flood storage capacity, if any, should it contain. For various assumed capacities, simulation models can be used to predict the impact on the downstream flood peaks. These hydraulic simulation models must include flood routing procedures from the reservoir to the downstream potential damage site and the flood control operating policy at the reservoir. For various downstream flood peaks, economic flood damages can be estimated. To calculate the expected annual damages associated with any upstream reservoir capacity, the probabilities of various damage levels being exceeded in any year need to be calculated.

The expected annual flood damage at a potential flood damage site can be estimated from an exceedance probability distribution of peak flood flows at that potential damage site together with a flow or stage damage function. The peak flow exceedance distribution at any potential damage site will be a function of the upstream reservoir flood storage capacity K_f and the reservoir operating policy. The computational process of determining the probability of exceedance of particular flood damages associated with a particular flood storage capacity and operating policy is illustrated graphically in Figure 11.22. The analysis requires three input functions that are shown in quadrants (a), (b) and (c). The dashed-line rectangles define point values on the three input functions in quadrants (a), (b) and (c) and the corresponding probabilities of exceeding a given level of damages in the lower right quadrant (d). The distribution in quadrant (d) is defined by the intersections of these dashed-line rectangles. This distribution defines the probability of equalling or exceeding a specified damage in any given year. The shaded area under the derived function is the annual expected damage, E[FD].

The relationships between flood stage and damage, and flood stage and peak flow, defined in quadrants (a) and (b) of Figure 11.22, must be known. These do not depend on the flood storage capacity in an upstream reservoir. The information in quadrant (c) defines the exceedance probabilities of each peak flow. Unlike the other two functions, this distribution depends on the upstream flood storage capacity and flood flow release policy. This peak flow probability of exceedance distribution is determined by simulating the annual floods entering the upstream reservoir in the years of record.

The difference between the expected annual flood damage without any upstream flood storage capacity and the expected annual flood damage associated with a flood storage capacity of K_f is the expected annual flood damage reduction. This is illustrated in Figure 11.23. Knowing the expected annual damage reduction associated with various flood storage capacities, K_f, permits the definition of a flood damage reduction function, $B_f(K_f)$.

If the reservoir is a single-purpose flood control reservoir, the eventual tradeoff is between the expected flood reduction benefits, $B_f(K_f)$, and the annual costs, $C(K_f)$, of that upstream reservoir capacity. The particular reservoir flood storage capacity that maximizes the net benefits, $B_f(K_f) - C(K_f)$, may be appropriate from a national economic efficiency perspective but it may not be best from a local perspective. Those occupying the potential damage site may prefer a specified level of protection from that

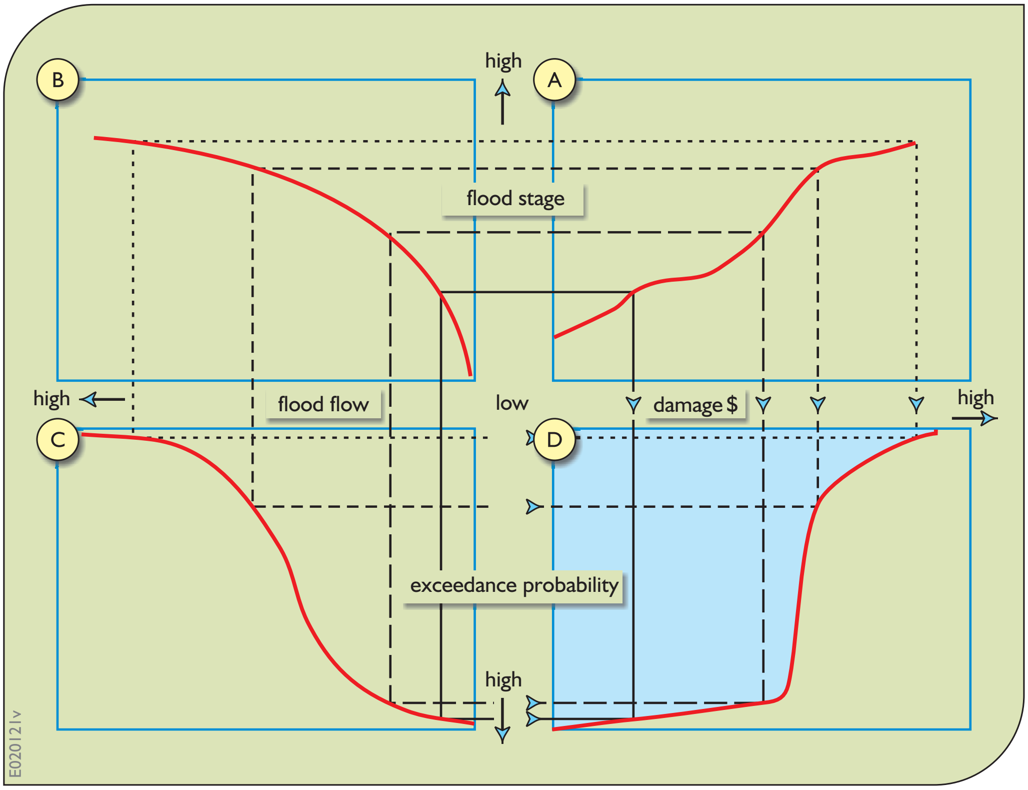

Figure 11.22. Calculation of the expected annual flood damage shown as the shaded area in quadrant (D) derived from the expected stage-damage function (A), the expected stage-flow relation (B), and the expected probability of exceeding an annual peak flow (C).

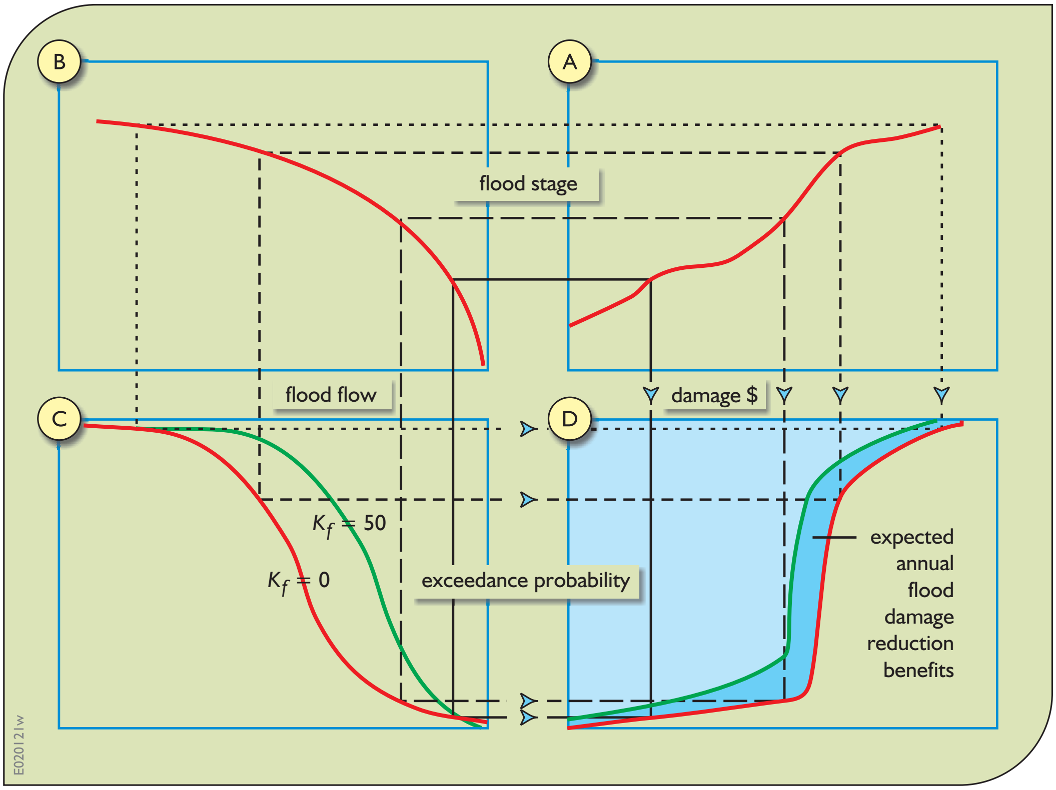

Figure 11.23. Calculation of expected annual flood damage reduction benefits, shown as the darkened portion of quadrant (D), associated with a specified reservoir flood storage capacity.

reservoir storage capacity, rather than the protection that maximizes expected annual net benefits, $B_f(K_f) - C(K_f)$.

If the upstream reservoir is to serve multiple purposes, say for water supply, hydropower and recreation as well as for flood control, then the expected flood reduction benefit function just derived could be a component in the overall objective function for that reservoir. Alternatively, the flood storage capacity, K_f, can be based on a particular predefined level of flood protection, regardless of the damage reduction. Again, both methods are discussed in greater detail in Appendix D.

Total reservoir capacity K will equal the sum of dead storage capacity K_d, active storage capacity K_a and flood storage capacity K_f, assuming they are the same in each period t. In some cases they may vary over the year. If the required active storage capacity can occupy the flood storage zone when flood protection is not needed, then the total reservoir capacity K will be the dead storage, K_d, plus the maximum of either the actual storage volume and flood storage capacity in the flood season, or the actual storage volume in non-flood season.

$K \geq K_d + S_t + K_f$ for all periods t in flood season plus the following period that represents the end of the flood season (11.74)

$K \geq K_d + S_t$ for all remaining periods t (11.75)

In the above equations, the dead storage capacity, K_d, is a known variable. It is included in the capacity Equations 11.74 and 11.75, assuming that the active storage capacity is greater than zero. Clearly, if the active storage capacity were zero, there would be no need for dead storage.

3.5.2. Channel Capacity

The unregulated natural peak flow of a particular design flood at a potential flood damage site can be reduced by upstream reservoir flood storage capacity, or it can be contained within the channel at the potential damage site by levees and other channel-capacity improvements. Of course both options may be applicable at some sites. What is needed is a function defining the reduction in the peak stage of some design flood and the cost of providing additional channel capacity, K_c. This function can then be included in a model along with the upstream flood storage capacity, K_f, to determine the best combination of alternative flood control measures that will protect the potential damage site from a predetermined design flood.

For example, consider a single-purpose flood control reservoir and channel capacity enlargement, either by dredging or dykes, whichever is cheaper, as alternatives for protecting from a design flood, Q_T, having a return period of T. Defining the functional relationship between K_f and peak flow reduction at the damage site, and defining K_c as the peak flow reduction due to channel capacity enlargement, then the least cost combination can be estimated from solving the following optimization model:

$$\text{minimize } \mathrm{Cost}_R(K_f) + \mathrm{Cost}_C(K_c) \tag{11.76}$$

subject to

$$Q_T \leq f_T(K_f) + K_c \tag{11.77}$$

Equations 11.76 and 11.77 assume that a decision will be made to provide protection from a design flood Q_T. It is only a question of how to provide the required protection.

Solving Equations 11.76 and 11.77 for peak flows Q_T of various design floods of return period T will identify the risk–cost tradeoff. This tradeoff function might look like what is shown in Figure 11.24.

3.6. Lake-Based Recreation

Recreation benefits derived from natural lakes as well as reservoirs are usually dependent on their storage levels. If docks, boathouses, shelters and other recreational facilities are installed on the basis of some assumed (target)

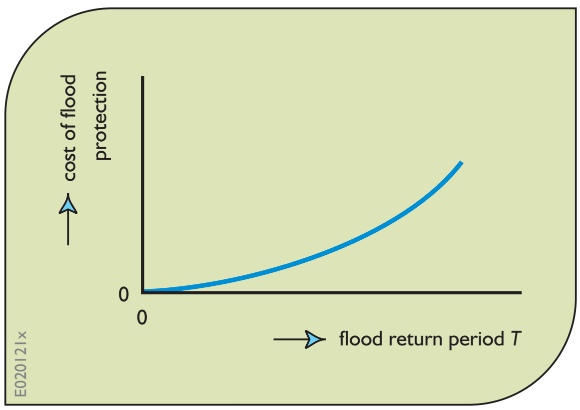

Figure 11.24. Tradeoff between minimum cost of flood protection and flood risk, as expressed by the expected return period.

lake level, and the lake levels deviate from the target value, there can be reduced recreational benefits. These storage targets and any deviations can be modelled in a way similar to Equation 11.66. The actual storage volume at the beginning of a recreation period t equals the target storage volume less any deficit or plus any excess;

$$S_t^s = T^s - D_t^s + E_t^s \tag{11.78}$$

The recreational benefits in any recreational period t can be defined on the basis of what they would be if the target were met, less the average of any losses that may occur from initial and final storage volume deviations, D_t^s or E_t^s, from the target storage volume in each period of the recreation season (Chapter 10).

4. River Basin Analysis

4.1. Model Synthesis

Each of the model components discussed above can be combined, as applicable, into a model of a river system. One such river system together with some of its interested stakeholders is shown in Figure 11.25.

One of the first tasks in modelling this basin is to identify the actual and potential system components and their interdependencies. This is facilitated by drawing a schematic of the system at the level of detail that will address the issues being discussed and of concern to these stakeholders. This schematic can be drawn over the basin as in Figure 11.26. The schematic without the basin is illustrated in Figure 11.27.

A site number must be assigned at each point of interest. These sites are usually where some decision must be made. Mass-balance and other constraints will need to be defined at each of those sites.

Figure 11.25. A multipurpose river system whose management is of concern to numerous stakeholders (with permission from *Engineering News Record*, McGraw Hill).

Figure 11.26. A schematic representation of the basin components and their interdependencies drawn over the map image of the basin.

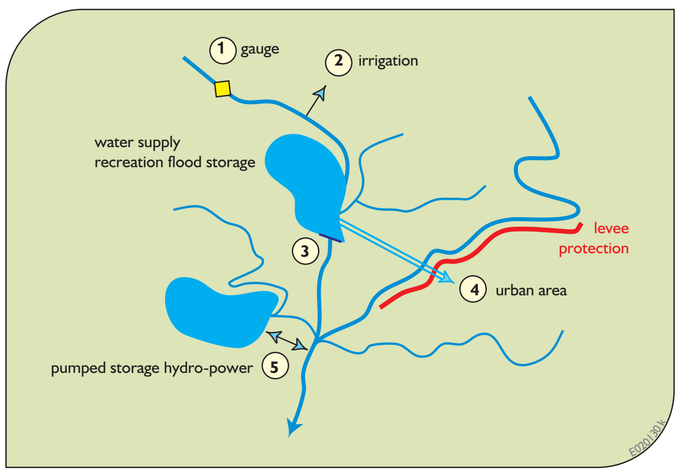

Figure 11.27. Schematic of river system showing components of interest at designated sites.

As shown in the schematic in Figure 11.27, this river has one streamflow gauge site, site 1, two reservoirs, sites 3 and 5, two diversions, sites 2 and 3, one hydropower plant, site 5, and a levee desired at site 4 to help protect against floods in the urban area. The reservoir at site 5 is a pumped storage facility. The upstream reservoir at site 3 is used for recreation, water supply, and flood control. The downstream reservoir is strictly for hydropower production.

Before developing a model of this river system, the number of within-year time periods t to include in the model and the length of each within-year time period should be determined. If a river system's reservoirs are to contain storage for the distribution of water among different years, called over-year storage, then a number of periods encompassing multiple years of operation must be included in the model. This will allow an evaluation of the possible benefits of storing excess water in wet years for release in dry years.

Many reservoir systems completely fill up almost every year, and in such cases one is concerned only with the within-year operation of the system. This is the problem addressed here. To model the within-year operation of the system, a year is divided into a number of within-year periods. The number of periods and the duration of each period will depend on the variations in the hydrology, the demands and the particular objectives, as previously discussed.

4.2. Modelling Approach Using Optimzation

Once the number and duration of the time steps to be modelled have been identified, the variables and functions used at each site must be named. It is convenient to use notation that can be remembered when examining the model solutions. The notation made up for this example is shown in Table 11.5. Economic efficiency objective function components are defined in Table 11.6.

The objective components listed in Table 11.6 are economic efficiency objectives. The *LD* and *LE* loss functions are associated with deficit and excess allocations, respectively. These and the other functions (whose meaning should be self evident) need to be known, of course. There could, and no doubt should, be other objective components defined as well, as discussed in Chapter 10. Nevertheless these objective components serve the purpose here of illustrating how a model of this river system can be constructed.

design and operating variables and parameters
all sites *s* and time periods *t*:
natural streamflows, based on gage flows at site 1, $Q^s(t)$
site 2
irrigation allocations, $X^2(t)$, for all periods t
annual irrigation target allocation, T^2
known fraction of annual irrigation target required for each period t, δ_t^2
irrigation diversion channel capacity, X^2
site 3
active initial storage volume. $S^3(t)$, in each period t
dead storage volume, Kd^3
reservoir release downstream, $R^3(t)$, in each period t
recreation storage volume target, T^3
deficit and excess storage volumes, $D^3(t)$ and $E^3(t)$, for each period t
flood storage volume capacity, Kf^3
total reservoir storage capacity, K^3
urban diversions, $X^3(t)$, in each period t
diversion capacity, X^3
site 4
channel flood flow capacity, Q^4
water supply target, T^4
known fraction of annual water supply target for each period t, δ_t^4
site 5
active initial storage volume, $S^5(t)$, in each period t
dead storage volume, KD^5
reservoir release through turbines, $QO^5(t)$, in each period t
quantity of water pumped back into reservoir, $QI^5(t)$, in each period t
energy produced, $EP^5(t)$, in each period t
energy consumed, $EC^5(t)$, in each period t
total storage capacity of reservoir, K^5
power plant/pump capacity, P^5
storage head function, $h(S^5(t))$, in each period t
average storage head, $H^5(t)$, in each period t

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Table 11.5. Names associated with required variables and functions at each site in Figure 11.27. The units of these variables and parameters, however defined, must be consistent.

The overall objective might be a weighted combination of all objective components such as:

$$\text{Maximize} \sum_s w_s NB^s \tag{11.79}$$

This objective function does not identify how much each stakeholder group would benefit and how much they would pay. Who benefits and who pays, and how much, may be important. If it is known how much of each of the

Table 11.6. Economic efficiency (net benefit) maximization objective components.

site 2
$NB^2 = \mathrm{IrrBenefit}^2(T^2) - \mathrm{Cost}_D(X^2)$
site 3
$NB^3 = \mathrm{RecBenefit}^3(T^3) - \sum_t [LD^3(D^3(t)) + LE^3(E^3(t))] + FBenefit^3(K_f^3) - Cost_K{}^3(K^3)$
site 4
$NB^4 = DivBenefit^4(T^4) - \sum_t [LD^4(D^4(t))] + FBenefit^4(Q^4) - Cost_Q(Q^4) - Cost_D(X^4)$
site 5
$NB^5 = EnergyBenefit^5(EP^5(t)) - EnergyCost^5(EC^5(t)) - Cost_p{}^5(P^5) - Cost_K{}^5(K^5)$

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net benefits derived from each site are to be allocated to each stakeholder group i, then these allocated portions, denoted as NB_i^s, of the total net benefits, NB^s, can be included in the overall objective:

$$\text{Maximize} \sum_i w_i \sum_s NB_i^s \tag{11.80}$$

Using methods discussed in Chapter 10, solving the model for various assumed values of these weights, w_s or w_i, can help identify the tradeoffs between different conflicting objectives, Equation 11.79, or conflicting stakeholder interests, Equation 11.80.

The next step in model development is to define the constraints applicable at each site. It is convenient to begin at the most upstream sites and work downstream. As additional variables or functions are needed, invent notation for them. These constraints tie the decision-variables together and identify the interdependencies among system components. In this example, Equations 11.81 through 11.97 (see Box 11.2), together with objective Equation 11.79 or 11.80, define the general structure of this river system model.

Before the model can be solved the actual functions must be defined. Then they may have to be made piecewise linear if linear programming is to be the optimization procedure used to solve the model. The process of defining functions may add variables and constraints to the model, as discussed in Chapters 4 and 10.

For T within-year periods t, this static model of a single year includes between $14T + 8$ and $16T + 5$ constraints, depending on the number of periods in the irrigation and recreation seasons. This number does not include the additional constraints that will surely be needed to define the functions in the objective function components and constraints. Models of this size and complexity, even though this is a rather simple river system, are usually solved using linear programming algorithms simply because other non-linear or dynamic programming (optimization) methods are more difficult to implement.

The model just developed is for a typical single year. In some cases it may be more appropriate to incorporate over-year as well as within-year mass-balance constraints, and yields with their respective reliabilities, within this modelling framework. This can be done as outlined in Sections 2.2.4 and 2.2.5 of this chapter.

The information derived from optimization models of river systems such as this one should not be considered as a final answer. Rather, it is an indication of system design and operation policies that should be further analysed using more detailed analyses. Optimization models of the type just developed serve as ways to eliminate inferior alternatives from further consideration rather than as ways of finding a solution that all stakeholders will accept as the best.

4.3. Modelling Approach Using Simulation

A simulation approach basically uses the same water balance equations and constraints as given in the text box.

Box 11.2. General Structure of the River System Model

At site 1

No constraints are needed at this gauge site.

At site 2

- The diverted water, $X^2(t)$, cannot exceed the streamflow, $Q^2(t)$, at that site.

$$Q^2(t) \geq X^2(t) \quad \forall t \text{ in the irrigation season} \quad (11.81)$$

- The diversion flow, $X^2(t)$, cannot exceed the diversion channel capacity, X^2.

$$X^2 \geq X^2(t) \quad \forall t \quad (11.82)$$

- The diversion flow, $X^2(t)$, must meet the irrigation target, $\delta_t^2 T^2$

$$X^2(t) \geq \delta_t^2 T^2 \quad \forall t \quad (11.83)$$

At site 3

- Storage volume mass balances (continuity of storage), assuming no losses.

$$S^3(t+1) = S^3(t) + Q^3(t) - X^3(t) - R^3(t) \quad \forall t, \quad T+1=1 \quad (11.84)$$

- Define storage deficits, $D^3(t)$, and excesses, $E^3(t)$, relative to recreation target, T^3.

$$S^3(t) = T^3 - D^3(t) + E^3(t) \quad \forall t \text{ in recreation season plus following period} \quad (11.85)$$

- Diverted water, $X^3(t)$, cannot exceed diversion channel capacity, X^3.

$$X^3(t) \leq X^3 \quad \forall t \quad (11.86)$$

- Reservoir storage capacity constraints involving dead storage, K_D^3, and flood storage, K_F^3, capacities.

$$S^3(t) \leq K^3 - K_D^3 - K_F^3 \quad \forall t \text{ in flood season plus following period}$$

$$S^3(t) \leq K^3 - K_D^3 \text{ for all other periods } t \quad (11.87)$$

At site 4

- Define deficit diversion, $D^4(t)$, from site 3, associated with target, $d_t^4 T^4$, if any.

$$X^3(t) = \delta_t^4 T^4 - D^4(t) \quad \forall t \quad (11.88)$$

- Channel capacity, Q^4, for peak flood flow, PQ_T^4, of specified return period T.

$$Q^4 \geq PQ_T^4 \quad (11.89)$$

At site 5

- Continuity of pumped storage volumes, involving inflows, $QI^5(t)$, and outflows, $QO^5(t)$, and assuming no losses.

$$S^5(t+1) = S^5(t) + QI^5(t) - QO^5(t) \quad \forall t \quad (11.90)$$

- Active storage capacity involving dead storage, K_D^5.

$$S^5(t) \leq K^5 - K_D^5 \quad \forall t \quad (11.91)$$

- Pumped inflows cannot exceed the amounts of water available at the intake; this includes the release from the upstream reservoir, $R^3(t)$, and the incremental flow, $Q^5(t) - Q^3(t)$.

$$QI^5(t) \leq Q^5(t) - Q^3(t) + R^3(t) \quad \forall t \quad (11.92)$$

- Define the energy produced, $EP^5(t)$, given the average storage head, $H(t)$, flow through the turbines, $QO^5(t)$, and efficiency, e.

$$EP^5(t) = (\text{const.})(H(t))(QO^5(t))e \quad \forall t \quad (11.93)$$

- Define the energy consumed, $EC^5(t)$, from pumping given the amount pumped, $QI^5(t)$.

$$EC^5(t) = (\text{const.})(H(t))(QI^5(t))/e \quad \forall t \quad (11.94)$$

- Energy production, $EP^5(t)$, and consumption, $EC^5(t)$, constraints given power plant capacity, P^5.

$$EP^5(t) \leq P^5 \text{ (hours of energy production in } t) \quad \forall t \quad (11.95)$$

$$EC^5(t) \leq P^5 \text{ (hours of pumping in } t) \quad \forall t \quad (11.96)$$

- Define the average storage head, $H^5(t)$, based on storage head functions, $h(S^5(t))$.

$$H^5(t) = (h(S^5(t+1) + h(S^5(t))/2 \quad \forall t \quad (11.97)$$

In a simulation approach, the analyst will have to specify the values of the decision variables (e.g. the diversion policies). The 'best' values of these decision-variables will have to be determined by sensitivity analyses, i.e. by changing these variables and seeing what the output will be in terms of the objective functions. Hence, simulation is a 'trial-and-error' approach.

A simulation model of the simple system as given in Figure 11.27 can easily be developed by means of a spreadsheet. More complicated systems can be simulated by using generic computer packages for river basin planning such as RIBASIM (WL | Delft Hydraulics, 2004), MIKE BASIN (DHI, 2003), and WEAP-21 (SEI, 2001). These river basin simulation packages support the development of a model schematization consisting of a network of nodes connected by links. The nodes represent reservoirs, dams, weirs, pumps, hydro-power stations, water users, inflows, artificial and natural bifurcations, intake structures, natural lakes and so on. The links transport water between the different nodes. Such a network represents the basin's features that are significant for the planning and management problem at issue. The network can be adjusted to provide the level of spatial and temporal detail required. The river basin is represented as a network schematization superimposed over a vector or raster map image.

Figure 11.28 shows the network schematization of RIBASIM of the simple river basin system of Figure 11.27. Within each time-step a water balance calculation is made in two phases:

1. The target setting phase, in which the water demands are determined (calculated from specific modules for irrigation, DMI, and other uses, or directly specified in volumes or discharges), resulting in targets for the releases from surface water reservoirs, aquifers and diversion flows at weirs and pumping stations.
2. The water allocation phase, in which the allocation to the users takes place according to the targets, availability of supply and allocation rules (priorities).

Water allocation to users can be implemented in a variety of ways. In its simplest format, water is allocated on a 'first come, first serve' principle along the natural flow direction. This allocation can be amended by rules which, for

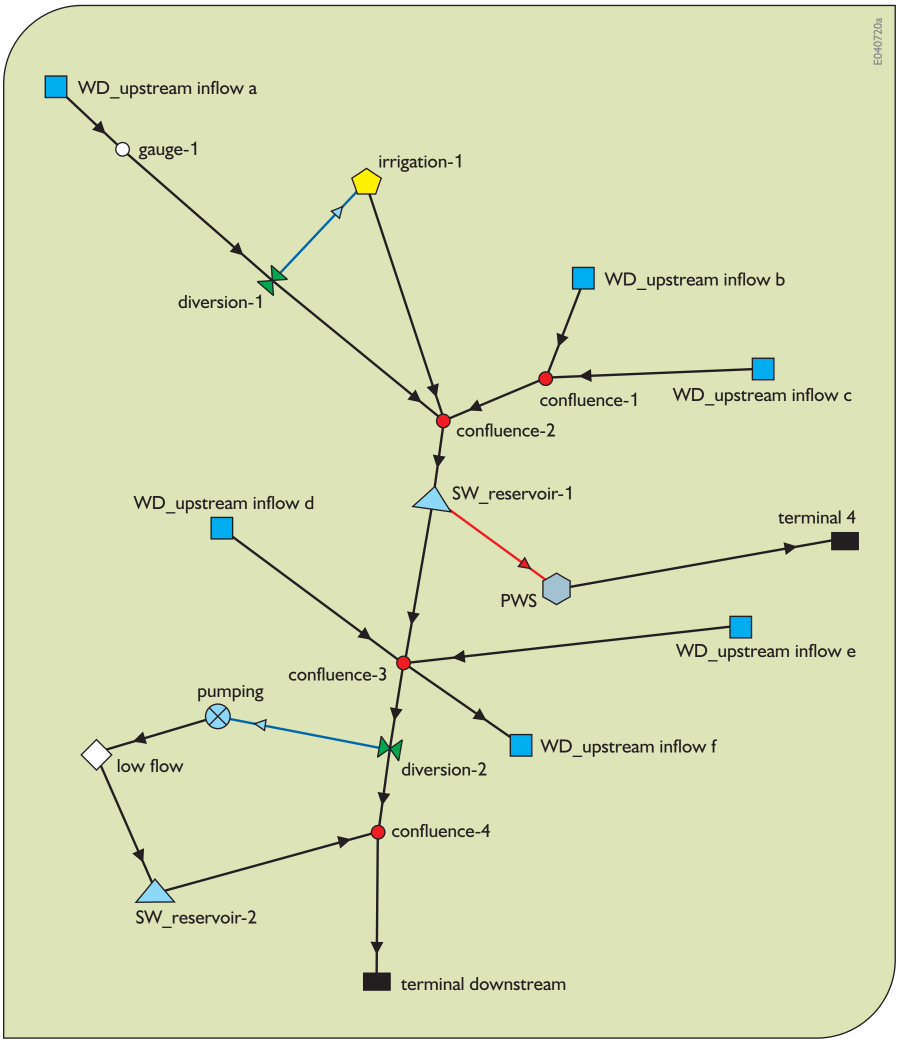

Figure 11.28. RIBASIM representation the river basin schematization of Figure 11.27.

example, allocate priority to particular users or result in allocations proportional to demands. On the basis of a set of simulations in which the values of important decision-variables are changed, thus defining a range of alternative development or management strategies, thusthe performance of the basin is evaluated in terms of water allocation, shortages, energy production, overall river basin water balance, crop yields, production costs and so on.

4.4. Optimization and/or Simulation

For determining the best design and operating policy variable values in river basin systems such as the one just illustrated, or even more complex ones, the use of both optimization and simulation models can be advantageous. As discussed in Chapter 3 (Section 2.2), optimization is often useful, not for finding the very best solution, but for eliminating the worst alternatives from further consideration. The remaining ones can then be analysed in greater detail, using more detailed simulation models. Simulation by itself begs the question: 'What to simulate?' Optimization by itself begs the question: 'Is the solution really the best?'

There is an art to using optimization as a preliminary screening tool. Some of this art is illustrated in Chapter 4, Section 5.3.

4.5. Project Scheduling

The river basin models discussed thus far in this chapter deal with static planning situations. Project capacities, targets and operating policies take on fixed values and one examines 'snapshot steady-state' solutions for a particular year in the future. These 'snapshots' only allow for fluctuations caused by hydrological variability. The non-hydrological world is seldom static, however. Demands and targets change in response to population growth, investment in agriculture and industry, and shifting priorities for water use. In addition, financial resources available for water resources investment are limited and may vary from year to year.

Dynamic planning models can aid those responsible for the long-run development and expansion of water resources systems. Although static models can identify target values and system configuration designs for a particular period in the future, they are not well adapted to long-run capacity expansion planning over a ten, twenty or thirty-year period. But static models may identify projects for implementation in early years that in later year simulations do not appear in the solutions. This was seen in some examples presented in Chapter 4.

This is a common problem in capacity expansion where each project has a fixed construction and implementation cost, as well as variable operating, repair and maintenance cost components. If there are two mutually exclusive competing projects, one may be preferred at a site when the demand at that site is low, but the other may be preferred if the demand is, as it is later projected to be, much higher. Which of the two projects should be selected now when the demand is low, given the uncertainty of the projected increase over time, especially assuming it makes no economic sense to destroy and replace a project already built?

Whereas static models consider how a water resources system operates under a single set of fixed conditions, dynamic expansion models must consider the sequence of changing conditions that might occur over the planning period. For this reason, dynamic expansion models are potentially more complex and larger than are their static counterparts. However, to keep the size and cost of dynamic models within the limitations of most studies, these models are generally restricted to very simple descriptions of the economic and hydrological variables of concern. Most models use deterministic hydrology and are constrained either to stay within predetermined investment budget constraints or to meet predetermined future demand estimates.

Dynamic expansion models can be viewed as network models for solution by linear or dynamic programming methods. The challenge in river system capacity expansion or project scheduling models is that each component's performance, or benefits, may depend on the design and operating characteristics of other components in the system. River basin project impacts tend to be dependent on what else is happening in the basin, that is, what other projects are present and how they are operated.

Consider a situation in which n fixed-scale discrete projects may be built during the planning period. The scheduling problem is to determine which projects to build or implement and in what order. The solution of this problem generally requires a resolution of the timing problem. When should each project be built or implemented, if at all?

For example, assume there are $n = 3$ discrete projects that might be beneficial to implement sometime over the next twenty years. This twenty-year period consists of four five-year construction periods y. The actual benefits derived from any new project may depend on the projects that already exist. Let $\boldsymbol{S}$ be the set of projects existing at the beginning of any construction period. Finally, let $NB_y(\boldsymbol{S})$ be the maximum present value of the total net benefits derived in construction period y associated with the projects in the set $\boldsymbol{S}$. Here, 'benefits' refers to any composite of system performance measures.

These benefit values for various combinations of discrete projects could be obtained from static river system models, solved for all combinations of discrete projects for conditions existing at the end of each of these four five-year periods. It might be possible to do this for just one or two of these four periods and apply applicable discount rates for the other periods. These static models can be similar to those discussed in the previous section of this chapter. Now the challenge is to find the sequencing of these three projects over the periods y that meet budget constraints and that maximize the total present value of benefits.

This problem can be visualized as a network. As shown in Figure 11.29, the nodes of this network represent the sets $\boldsymbol{S}$ of projects that exist at the beginning of the construction period. For these sets $\boldsymbol{S}$, we have the present value of their benefits, $NB_y(\boldsymbol{S})$, in the next five years. The links represent the project or projects implemented in that construction period. Any set of new projects that exceeds the construction funds available for that period is not shown on the network. Those links are infeasible. For the purposes of this example, assume it is not financially feasible to add more than one project in any single construction period. Let C_{ky} be the present value of the cost of implementing project k in construction period y.

The problem is to find the best (maximum benefits less costs) paths through the network. Each link represents a net benefit, $NB_y(\boldsymbol{S})$, over the next five-years obtained from the set of projects, $\boldsymbol{S}$, that exist less the cost of adding a new project k.

Using linear programming, define a continuous non-negative unknown decision-variable X_{ij} for each link between node i and node j. It will be an indicator of whether a link is on the optimum path or not. If after solving the model its value is 1, then the link connecting nodes i and j represents the decision to make in that construction period. Otherwise, its value is 0 indicating the link is not on the optimal path. The sequence of links having their X_{ij} values equal to 1 will indicate the most beneficial sequence of project implementations.

Let the net benefits associated with node i be designated NB_i (which equals the appropriate $NB_y(\boldsymbol{S})$ value), and C_{ij} the cost, C_{ky}, of the new project k associated with that link. The objective is to maximize the present value of net benefits less project implementation costs over all periods y.

$$\text{Maximise} \sum_i \sum_j (NB_i - C_{ij}) X_{ij} \tag{11.98}$$

Subject to:
Continuity at each node:

$$\sum_h X_{hi} = \sum_j X_{ij} \text{ for each node } i \text{ in the network.} \tag{11.99}$$

Sum of all decision-variable values on the links in any one period y must be 1. For example, in period 1:

$$X_{00}+X_{01}+X_{02}+X_{03}=1 \tag{11.100}$$

The sums in Equation 11.99 are over nodes h having links to node i and over nodes j having links from node i.

The optimal path through this network can also be solved using dynamic programming. (Refer to the capacity expansion problem illustrated in Chapter 4). For a backward-moving solution procedure, let

$\boldsymbol{s}$ = subset of projects k not contained in the set $\boldsymbol{S}$ ($\boldsymbol{s} \notin \boldsymbol{S}$).

$\$_y$ = the maximum project implementation funds available in period y.

$F_y(\boldsymbol{S})$ = the present value of the total benefits over the remaining periods, $y, y+1, \ldots, 4$.

$F_Y(\boldsymbol{S}) = 0$ for all sets of projects S following the end of the last period.

The recursive equations for each construction period, beginning with the last period, can be written

$$F_y(\boldsymbol{S}) = \underset{\boldsymbol{s} \in S}{\text{maximum}} \{NB_y(\boldsymbol{S}) - \sum_{k \in \boldsymbol{s}} C_{ky} + F_{y+1}(\boldsymbol{S} + \boldsymbol{s})\} \; \forall \boldsymbol{S} \tag{11.101}$$

$$\sum_{k \in \boldsymbol{s}} C_{ky} \leq \$_y$$

Defining $F'_y(\boldsymbol{S})$ as the present value of the total benefits of all new projects in the set $\boldsymbol{S}$ implemented in all periods up

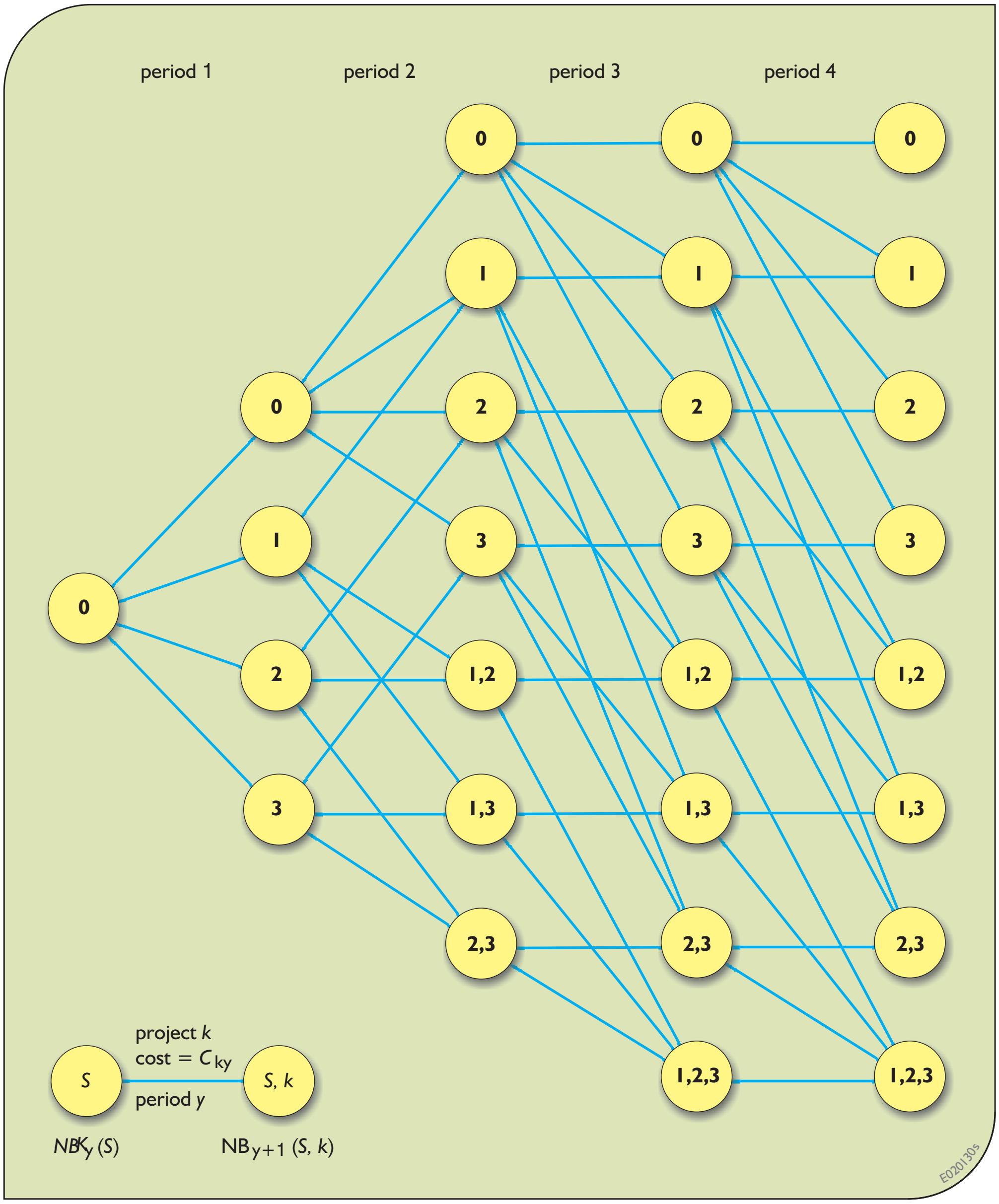

Figure 11.29. Project scheduling options. Numbers in nodes represent existing projects. Links represent new projects, the difference between the existing projects at both connecting nodes.

to and including period y, and the subset s of projects k being considered in period y now belonging to the set $\boldsymbol{S}$ of projects existing at the end of the period, the recursive equations for a forward-moving solution procedure beginning with the first period, can be written:

$$F_y(\boldsymbol{S}) = \underset{\boldsymbol{s} \in \boldsymbol{S}}{\text{maximum}} \{NB_y(\boldsymbol{S}) - \sum_{k \in \boldsymbol{s}} C_{ky} + F_{y+1}(\boldsymbol{S} + \boldsymbol{s})\} \; \forall \boldsymbol{S} \tag{11.102}$$

$$\sum_{k \in \boldsymbol{s}} C_{ky} \leq \$_y$$

where $F_0'(0) = 0$.

Like the linear programming model, the solutions of these dynamic programming models identify the sequencing of projects recognizing their interdependencies. Of interest, again, is what to do in this first construction period. The only reason for looking into the future is to make sure, as best as one can, that the first period's decisions are not myopic. Models like these can be developed and solved again with more updated estimates of future conditions when next needed.

Additional constraints and variables might be added to these scheduling models to enforce requirements that some projects precede others, or that if one project is built another is infeasible. These additional restrictions usually reduce the size of a network of feasible nodes and links, as shown in Figure 11.29.

Another issue that these dynamic models can address is the sizing or capacity expansion problem. Frequently, the scale or capacity of a reservoir, pipeline, pumping station or irrigation project is variable and needs to be determined concurrently with the solution of the scheduling and timing problems. To solve the sizing problem, the costs and capacities in the scheduling model become variables.

5. Conclusions

This chapter on river basin planning models has introduced some ways of modelling river basin components, separately and together within an integrated model. Ignored during the development of these different model types were the uncertainties associated with the results of these models. As discussed in Chapters 7 through 9, these uncertainties may have a substantial effect on the model solution and the decision taken.

Most of this chapter has been focused on the development of simplified screening models, using simulation as well as optimization methods, for identifying what and where and when infrastructure projects should be implemented, and of what capacity. The solution of these screening models, and any associated sensitivity and uncertainty analyses, should be of value prior to committing to a more costly design modelling exercise.

Preliminary screening of river basin systems, especially given multiple objectives, is a challenge to accomplish in an efficient and effective manner. The modelling methods and approaches discussed in this chapter serve as an introduction to that art.

6. References

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