4.1 WATER RESOURCE ASSESSMENT AND WATER PROJECTS [HOMS A00]

4.1.1 The need for water resource assessment

Water resource assessment is the determination of the sources, extent, dependability and quality of water resources, which is the basis for evaluating the possibilities of their utilization and control (UNESCO/WMO, 1997). Water resource assessment is of critical importance to sound and sustainable management of the world's water resources. Several reasons for this may be cited (WMO/UNESCO, 1991):

(a) The world's expanding population is placing increasing demands on water for drinking, food production, sanitation and other basic social and economic needs, but the world's water resources are finite. The rising demand has reached its limit in some areas and will reach the limit in many other areas within the next two decades. Should present trends continue, the world's water resources will be fully utilized before the end of the next century;

(b) Human activities are becoming increasingly intensive and diverse, producing a definite, ever-growing impact on natural resources through depletion and pollution. This is particularly the case for water, whose quality for many purposes can be severely degraded by physical changes and by pollution caused by a wide range of chemicals, microorganisms, radioactive materials and sediments;

(c) Water-related natural hazards, such as floods, droughts, and tropical cyclones, cause immeasurable destruction of human life and property, and have so during the course of history. Deforestation and urbanization, in particular, have exacerbated flood hazards by increasing the magnitude and frequency of floods;

(d) There is a growing realization that the world's climate is not constant, and indeed may well be changing in response to human activity. While the relationship between increased global temperatures and greenhouse-gas-induced warming has been widely publicized, more attention should be paid to the effects of climate on the distribution of rainfall, runoff, and groundwater recharge, which are likely to be significant. It cannot be assumed that the patterns of these hydrological phenomena will not change.

Effective water management can be achieved through sound decision-making based on reliable data and information on the status and trends of water resources, including quantity, quality, statistics on events such as floods, for example, and use for human purposes.

Water resource assessment is generally a prerequisite for water resources development and management, as recognized as early as 1977 by the United Nations Water Conference held in Mar del Plata (Resolution 1 and Recommendation A of the Mar del Plata Action Plan). The Conference stressed the need for greater knowledge about the quantity and quality of surface-water and groundwater resources, and for comprehensive monitoring to guide the management of these resources. Furthermore, the International Conference on Water and the Environment, held in Dublin from 26 to 31 January 1992, recommended a number of actions in support of national water resource assessment (United Nations, 1992).

4.1.2 Water resource assessment programme components

In order to permit a preliminary assessment of available water resources on which to base national or regional long-term plans for overall water resources development, a basic water resource assessment programme involves the collection and processing of existing hydrological and hydrogeological data, plus the auxiliary data required for their areal interpolation.

These plans should be based on or geared to present and future water needs. The components of a water resource assessment programme are shown, in Figure II.4.1, and are mainly as follows (UNESCO/WMO, 1997):

(a) The collection of hydrological data – historical data on water cycle components at a number of points distributed over the assessment area;

(b) The collection of physiographic data – data on the natural characteristics of the terrain that determine the areal and time variations of the water cycle components, such as topography,
soils, surface and bed rock geology, land use and land cover;
(c) The techniques used for the areal assessment of water resources – techniques for converting data into information and for relating the hydrological data to the physiographic data to obtain information on water resource characteristics at any point of the assessment area.

A basic water resource assessment programme is considered adequate if these three components are available and if, by relating them, they are sufficiently accurate to supply the water resources information required for planning purposes at any point of the assessment area. The country concerned will need to define the type of information required for planning, the manner in which this information is produced and transmitted to users and the effects of a lack of or inaccurate information on the decision-making process at the planning stage.

All basic water resource assessment activities require skilled personnel, sound equipment and techniques for field surveys, network design and operation, and development of reliable areal interpolation techniques. This, in turn, may require training and education of the required personnel, and basic and applied research to develop the required technology. An analysis of these activities can provide indications of their adequacy for the purpose of basic water resource assessment or, if inadequate, the additional means to be devoted to them to provide the required base for future development of an adequate water resource assessment programme.

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**Figure II.4.1. Components of a basic water resource assessment programme**

- **Collection of hydrological data** – water cycle components, including quantity and quality of surface and groundwater
- **Collection of physiographic data** – topographic, soils, geology
- **Techniques of areal assessment of water resources** – regionalization techniques
- **Water-resources information** – data banks, maps
- **Users** – planning, design and operation of water resources facilities
- **Basic and applied research**
- **Education and training**
4.1.3 Evaluation of water resource assessment activities

Water resource assessment is a national responsibility, and any evaluation of the extent to which it is being undertaken adequately in a country is also the responsibility of the country concerned. The WMO/UNESCO Water Resources Assessment: Handbook for Review of National Capabilities (UNESCO/WMO, 1997) was prepared with the aim of increasing the capabilities of countries to evaluate their achievements in water resource assessment and to provide a general framework for determining their needs and the actions necessary to achieve minimum requirements. The methodology proposed in the Handbook comprises the full range of topics and activities that are included in a water resource assessment programme. The current levels of basic water resource assessment are compared with minimum acceptable requirements in terms of installations, equipment, skilled personnel, education, training and research. It contains detailed checklists for each component (see Figure II.4.1) and offers advice as to how each activity might be evaluated, in most cases in quantifiable terms.

The results of the evaluation will be different for each country, depending on the characteristics of the corresponding basic water resource assessment programme and the country's characteristics and needs. Nevertheless, a minimum set of results is expected in practically each case. This set includes the following items:

(a) An analysis of the existing institutional framework for carrying out a basic water resource assessment programme with its resulting advantages, disadvantages and related constraints;
(b) A comparative evaluation of the measurement networks and indications of network elements that require improvement with respect to station density, equipment, operational and supervisory staff, and other factors;
(c) A review of the available surveys and programmes for collecting and processing physiographic data pertinent to basic water resource assessment;
(d) An evaluation of the application of various techniques for areal extension of basic water resource assessment and related data- and information-transfer techniques;
(e) An analysis of the hydrological information requirements for long-term planning, of the production and flow of this information to the user, and of the results of the use of such information in the planning process, which demonstrates the basic water resource assessment programme's adequacy or inadequacy;
(f) An estimation of the personnel and skills required for an adequate basic water resource assessment programme and appraisal of existing education and training programmes compared with current and future requirements;
(g) A review of basic and applied research activities in the country and region, their adequacies or inadequacies for water resource assessment compared with current and future needs, including needs for regional and international scientific and technological cooperation;
(h) Definition of the major gaps in the programme with regard to institutional framework, financial resources, instrumentation, techniques and others;
(i) Recommendations for eliminating inadequacies of the basic water resource assessment programme through national or regional cooperation and/or international aid.

4.1.4 Water projects

Water is needed in all aspects of life. The overall objective of water resource management is to make certain that adequate supplies of good-quality water are available for the population and various socio-economic developments of the society, while preserving the hydrological and biochemical functions of the ecosystem. There is a growing awareness that development, including water resources development, must be sustainable. This implies that the world’s natural resources must be managed and conserved in such a way as to meet the needs of present and future generations.

This chapter provides guidance on the application of the hydrological analysis methods described in Chapters 5, 6 and 7 for the design and operation of water management projects in order to meet the above-mentioned objective. In addition to the analysis to be undertaken as described in this chapter, a number of social, economic and environmental considerations should also be taken into account; however, these are beyond the scope of this Guide.

4.1.5 Purposes served by a water management project

As explained in Chapter 3, an integrated approach to river basin planning and management is suitable for handling the cross-sectoral activities. The holistic management of freshwater as a finite and vulnerable resource, and integration of the sectoral water plans and programmes within the national economic and social policy, are of paramount importance. Consideration of equitable and
responsible use of water is central to addressing the United Nations Millennium Development Goals and eradicating poverty.

The natural water cycle is spatially and temporally complex, and yet fulfilling human needs requires a stable water supply. It is therefore essential to implement water resources development and management strategies which generally involve some form of engineering intervention. Pressures on water systems due to growth in population and economic development have made it imperative that the engineering analyses needed for water development projects be more impartial and scientific based than in the past.

A water management project may serve one or more of the following objectives:
(a) Municipal water supply;
(b) Irrigation;
(c) Industrial water supply;
(d) Groundwater management;
(e) Power generation;
(f) Flood management;
(g) Navigation;
(h) Recreation, aesthetics and tradition;
(i) Salinity and sediment control;
(j) Pollution abatement;
(k) Fish and wildlife conservation;
(l) Other environmental considerations.

4.1.6 Multi-purpose projects
With the increasing level of development and use of water resources throughout the world, it is becoming ever more important to plan projects that can serve a number of purposes simultaneously. For example, a planned storage reservoir may provide both water supply and flood control downstream. Hydrological data required for the design of a multi-purpose project are basically an aggregate of the data required for the various single purposes involved. The methods of analysis, although similar to those applied in design of single-purpose projects, are more complex. A series of plans involving combinations of project sizes and methods of operation must be made to determine the optimum plan.

Conflicts can arise when attempting to manage water resources for a number of needs. The challenge of designing and operating systems to serve multiple functions is discussed in 4.2.

4.1.7 Project cycle
The project cycle is illustrated in Figure II.4.2. The cycle starts with the identification process, in which the following questions should be answered:
(a) Is the project technically feasible?
(b) Will total benefits exceed costs?
(c) Who benefits?

Figure II.4.2. Project cycle
(d) Are there better alternatives?
(e) Are social and environmental costs accepted?

Project preparation calls for a clear definition of project goals and objectives. Once they are defined, it is possible to identify relevant problems and feasible solutions. In addition, it is necessary to analyse pertinent data and information.

4.1.8 Preliminary investigation of water management projects

Before appreciable expenditure of time and money can be justified for the planning of a water management project, a preliminary investigation must be made of its feasibility, desirability, scope and its possible effect on those hydrological factors that influence the environment and the efficiency of other projects. Although the investigation has to be based on whatever material may be available, for example fragmentary hydrological records, old maps and reports, it must be carried out with great care because it is at this stage that conceptual planning decisions are often made and that important aspects and consequences of the project may become apparent. If the preliminary investigation indicates that the project potential is favourable, then more detailed studies would be initiated.

The types of hydrological data required for water management are given in Table II.4.1 below.

Table II.4.1. Data required for water management

<table>
<thead>
<tr>
<th>Purpose</th>
<th>Features</th>
<th>Concern</th>
<th>Required data</th>
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</thead>
<tbody>
<tr>
<td>Reconnaissance</td>
<td>Hydrology</td>
<td>Drainage network</td>
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<td>Watersheds</td>
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<td>Springs</td>
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<td>Distinction of perennial from intermittent</td>
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<td>and ephemeral streams</td>
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<td>Physiography</td>
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<td>Topography and morphology</td>
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<td>Soil cover and types</td>
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<td>Urbanization</td>
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<td>Meteorology</td>
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<td>Temperature distribution</td>
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<td>Wind distribution</td>
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<td>Snowpack distribution</td>
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<td>Streamflow</td>
<td>1, 2, 3, 4, 7, 8, 9 – at selected sites</td>
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<td>Floods</td>
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<td>Groundwater</td>
<td>12, 13</td>
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<td>Flood control</td>
<td>Structures</td>
<td>Water level</td>
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<td>Depth–discharge relationship for important</td>
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<td>points</td>
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<td>Hydraulic–topographic relationships in the</td>
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<td>flood plain</td>
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<td>Flood-plain occupancy</td>
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<td>Rainfall</td>
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<td>Statistics of heavy rainfall in the general</td>
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<td>area under consideration</td>
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<td>Pairs of floods and their causing precipitation</td>
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<td>Flood warning</td>
<td>Forecast</td>
<td>Travel times of floods</td>
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<td>Time lag between precipitation and runoff</td>
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<td>Flood synchronization at different tributaries</td>
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<td>Prediction</td>
<td>Time series of floods</td>
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<td></td>
<td>Time series of heavy precipitation</td>
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<td>Flood zoning and insurance</td>
<td>Flood extent</td>
<td>Area–duration–frequency of floods</td>
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<td>Scour and sedimentation by floods</td>
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<tr>
<td>Purpose</td>
<td>Features</td>
<td>Concern</td>
<td>Required data</td>
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<td>Irrigation</td>
<td>Demand</td>
<td>Precipitation</td>
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<td>Evapotranspiration</td>
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<td>Soil moisture</td>
<td>Soil type</td>
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<td>Supply</td>
<td>Streamflow</td>
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<td>Groundwater</td>
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<td></td>
<td>Reservoir</td>
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<td>Groundwater management, including recharge</td>
<td>Aquifers</td>
<td>Groundwater</td>
<td>12, 13</td>
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<td></td>
<td>Reservoirs and ponds</td>
<td>Streamflow</td>
<td>1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11</td>
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<td></td>
<td>Bank infiltration</td>
<td>Streamflow</td>
<td>3, 4, 6, 7, 8, 9</td>
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<td></td>
<td>Wells</td>
<td>Streamflow</td>
<td>1, 2, 3, 4, 5, 6, 8, 9, 10, 11</td>
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<td>Power generation</td>
<td>High-head dams</td>
<td>Streamflow</td>
<td>1, 2, 3, 4, 5, 6, 8, 10, 11</td>
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<td></td>
<td>Low-head dams</td>
<td>Streamflow</td>
<td>2, 3, 4, 6, 7, 8</td>
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<tr>
<td>Navigation</td>
<td>Channels</td>
<td>Water depth</td>
<td>Depth–discharge relationship for important points</td>
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<td>Flood flows</td>
<td>4, 6</td>
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<td>Rates of high water rise</td>
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<td>Time lag between rises at different points along the streams</td>
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<td>Time lag from heavy precipitation to high water</td>
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<td>Snowmelt distribution</td>
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<tr>
<td>Municipal supply</td>
<td>Rivers</td>
<td>Streamflow and springflow</td>
<td>1, 2, 3, 4, 7, 9</td>
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<td></td>
<td>Reservoirs</td>
<td>Streamflow</td>
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<td>Groundwater</td>
<td>12, 13</td>
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<tr>
<td>Industrial use</td>
<td>Rivers</td>
<td>Streamflow</td>
<td>1, 2, 3, 4, 7, 8, 9</td>
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<td></td>
<td>Reservoirs</td>
<td>Streamflow</td>
<td>1, 2, 3, 4, 5, 6, 8, 9, 10, 11</td>
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<tr>
<td></td>
<td>Aquifers</td>
<td>Groundwater</td>
<td>12, 13</td>
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</tbody>
</table>
Various applications for water management are discussed in the following sections of this chapter: 4.2 provides information about estimating yield and fixing reservoir capacity; 4.3 is devoted to flood management; 4.4 to irrigation and drainage; 4.5 to hydropower and energy-related projects; 4.6 to navigation and river training; 4.7 to urban water resources management; 4.8 to sediment transport and river channel morphology; 4.9 and 4.10 are devoted to environmental issues.

### 4.2 ESTIMATING RESERVOIR CAPACITY AND YIELD [HOMS K75]

#### 4.2.1 General

This section addresses the yields achievable and the storages required to maintain certain levels of yield, with respect to water resource systems. The focus is on surface water, although the water resources practitioner should always be aware of...
the hydrological interdependencies between surface water and groundwater which simply constitute different occurrences of the same resource in the hydrological cycle. Most principles are explained with respect to single river and single reservoir systems, although approaches for dealing with complex water resource systems comprising multiple reservoirs in different basins are also addressed.

The yield from a water resource system is the volume of water that can be abstracted at a certain rate over a specific period of time, generally expressed as an annual volume, such as million m$^3$ per year. The rate at which water needs to be abstracted may vary throughout the year, depending on the intended use. For domestic, industrial and mining uses, water is required at a relatively constant rate throughout the year, whereas strong seasonality occurs with respect to irrigation. Natural streamflow, in contrast, is much more variable. Rivers typically display strong seasonality in their natural runoff, compounded by within-season fluctuations in flow as well as large variations in total annual runoff.

If a constant abstraction rate is considered, the highest yield that can be abstracted from an unregulated river is equal to the lowest flow in the river as demonstrated in Figure II.4.3. By regulating streamflow by means of dams, water can be stored during periods of high flow for release during periods of low flow, as shown by the dotted lines on the diagram. This increases the rate at which water can be abstracted on a constant basis and, consequently, the yield. The greater the storage, the greater the yield that can be abstracted within the constraints of the physical characteristics of the system. Larger annual yields can also be obtained where seasonal demand patterns show good correlation with the streamflow characteristics. For ease of description, a constant abstraction rate is used in the remainder of the chapter, unless otherwise specified.

In areas where the average annual streamflow, or mean annual runoff, is well in excess of water requirements, but where the minimum streamflow may drop below the required abstraction rate, the focus is typically on determining the reservoir capacity required for bridging the period of low flow in order to maintain the desired yield. As streamflow varies from year to year, low flows (similar to floods) are not always of the same severity and duration. The amount of water that can be abstracted without failure, the yield, consequently also varies from year to year. Consideration must therefore also be given to the economics of whether sufficient storage should be provided to maintain the yield even under the most severe low flow (drought) conditions, or whether it would be more economical to provide less storage and thereby accept some degree of failure from time to time, with respect to supplying the full yield. Thus the challenge is to weigh the expected benefits against the risk, the associated costs of failure and the cost of storage.

Where streamflow is limited or where a water resource is already highly utilized, the focus shifts to the optimal utilization of water and the yield achievable from different storage capacities, rather than determining the storage required to maintain a desired yield. In such cases resource optimization adds an additional dimension to the initial problem of risk versus cost optimization.

With the high degree of water resources development and utilization already existing in most parts of the world, resource optimization is becoming increasingly important. Due attention is therefore given to aspects of resource optimization in the descriptions that follow.

4.2.2 Concepts of yield

This section presents the main parameters and key concepts related to the determination of yield, as referred to in this chapter.

4.2.2.1 Sequence

A sequence is defined as a chronological time series of data such as streamflows, precipitation and evaporation for a particular location. Monthly data are most commonly used with respect to the determination of yields for large water resource systems, although longer or shorter intervals may be selected.
As can be interpreted from Figure II.4.3, the configuration of the streamflow sequence has a determinant influence on the yield that can be obtained from a river. Should the streamflow be constant, for example, a yield equal to the average flow can theoretically be abstracted without any flow regulation. The larger the variability of streamflow, the greater the requirements for regulation through storage.

4.2.2.2 Natural influences

Losses from rivers, such as those due to evaporation and infiltration, are part of the hydrological cycle and are reflected in observed streamflow records used as a basis for the determination of yield. The creation of large water bodies as a means of flow regulation changes the pre-project natural equilibrium conditions, normally resulting in additional evaporation and infiltration.

4.2.2.2.1 Evaporation

The depth of water that evaporates annually from a reservoir surface may vary from about 400 mm in cool, humid climates to more than 2 500 mm in hot, arid regions. Therefore, evaporation is an important consideration in many projects and deserves careful attention.

Methods for estimating reservoir evaporation from pan observations and meteorological data are described in Volume I, 4.2, of this Guide. In the absence of pan evaporation or other appropriate meteorological observations at or near the reservoir site, regional estimates of these quantities are used to assess reservoir evaporation.

It is important that the net evaporation be taken into account in the water balance calculations (see Volume I, 4.2.3), thus allowing for precipitation on the reservoir surface. As periods of high and low streamflow normally correspond to wet and dry climatological conditions, the most realistic representation of the actual conditions is obtained where the precipitation or evaporation and streamflow sequences are fully synchronized. Should corresponding evaporation data not be available, as often occurs in practice, average monthly data could be used, at least to reflect seasonal variations.

Evaporation from reservoir surfaces not only represents a loss in potential yield, but also a loss of water from the surface water resource system, and thus a net loss of resource. Specific attention should therefore be given to the minimization of evaporation. Wherever practicable, the minimum storage surface area per unit volume of storage should be sought in the selection of dam sites. Extensive research has been conducted into evaporation suppression by the spreading of monomolecular films on water surfaces (see Volume I, 4.4.1), but practical problems in the application of these techniques to large storages still remain unsolved. Thermal stratification in reservoirs and the temperature difference between inflow and outflow can have a significant impact on reservoir evaporation. These influences are difficult to reliably quantify from theory and can best be judged from comparisons to existing storages.

4.2.2.2.2 Infiltration

Losses from reservoirs due to infiltration or seepage are highly dependent on local hydrogeological conditions.

Dam sites are mostly selected where foundation conditions are good and the geological formations underlying the reservoir basin are relatively impermeable. In such cases, groundwater levels in the vicinity of a new reservoir generally stabilize around a new equilibrium some time after the new reservoir has been brought into operation, with relatively small and even insignificant losses due to infiltration.

Where a reservoir basin or a part thereof is underlain by permeable strata such as sands, dolomite and karst formations, the losses can be significant. Measures to control such losses can be technically difficult and expensive to implement, and can render a project unfeasible.

Estimates of expected infiltration and seepage losses may be derived from geological investigations of the reservoir basin and dam site, and from comparisons with existing reservoirs in similar conditions. Unlike losses caused by evaporation, those caused by infiltration and seepage do not necessarily constitute a net loss of resource as they may contribute to groundwater recharge or to discharge downstream from a control structure.

4.2.2.2.3 Sedimentation

Sediment deposition in reservoirs reduces the available storage over time and therefore can have an impact on the long-term yield from a reservoir and the feasibility of a reservoir design project.

Any reservoir design must therefore account for the volume of sediment expected to accumulate over
the economic life of a dam and to provide for this through an equivalent volume of additional storage in the design of the reservoir or through an assumed decrease in the storage, and therefore the yield, over time. If it has deposited sediment in a reservoir, the river is likely to erode its downstream channel more than previously, and this should be taken into account during early planning stages.

It is essential to develop a bathymetric survey monitoring programme to ensure that expected sediment accumulations are consistent with those that occur in reality. This is especially true in regions where sediment transport is episodic and linked to unpredictable and extreme hydrological events, such as in semi-arid and arid areas, and also where catchment land-use changes have increased sediment production. Bathymetric surveys can be undertaken using standard water depth sounding methods, or through appropriate remote-sensing approaches. Depending on the sediment load and grain size distribution, as well as the streamflow and reservoir basin characteristics, much of the sediment deposition may occur in the upper reaches of a reservoir basin. This makes it expensive or difficult to remove the sediment. However, it is possible to design outlet works (scour gates) that can be used periodically to scour some of the accumulated sediment from a reservoir basin. More detail on sediment discharge, transport characteristics and the possible scouring of sediment can be obtained from Volume I, Chapter 5, and Volume II, 4.8.

4.2.2.3 Human influences

Water resources developments and some land-use activities upstream of a project site alter the natural streamflow characteristics at the project and can have significant impacts on its yield. Water resources developments can include regulation structures, diversions, abstractions, return flows and transfers from other catchments. Land-use activities with the greatest impact on water resources and sediment loads are as follows: urbanization, afforestation, deforestation, cultivation of certain crops such as rice and sugar, denudation of land and some forms of rainwater harvesting. The subsequent sections of this chapter relate to such activities and provide advice that is of use in assessing the likely impact of human influences.

It is important that human influences be properly accounted for in determining the yield from a water resource system. In particular, any trends should be noted, and due consideration be given to possible future developments.

4.2.2.4 Observed streamflow

Observed or actual streamflow sequences refer to the streamflow data as recorded in the field (see Volume I, Chapter 5). Therefore, such records inherently reflect the impacts of human influences and, with the exception of natural, or virgin, catchments, show some variations over time. In general, observed sequences require some processing for the infilling of missing data (see Volume I, 9.7.2) and to account for the impacts of development.

4.2.2.5 Naturalized streamflow sequences

Naturalized streamflow sequences are representative of the streamflow conditions prior to influences by humankind. In totally undeveloped catchments, the observed streamflows reflect the natural conditions perfectly. For catchments where development has occurred, realistic estimates can be made of what the streamflows would have been under natural conditions by calculating the impacts of the various influencing factors and adjusting the observed streamflow sequences accordingly.

4.2.2.6 Synthetic streamflow sequences

A synthetic streamflow sequence is one that is artificially produced by using a computer model. Two kinds of synthetic sequences are used with respect to streamflow: deterministic sequences and stochastic sequences.

Deterministic sequences are mainly used to fill in and extend incomplete streamflow sequences. This is achieved through the use of hydrological models, as described in Chapter 6.

A stochastic sequence is one that randomly varies in time, possibly with some dependence structure, and purports to offer alternatives to the observed sequence as a means of assessing what might plausibly be experienced in future (Box and Jenkins, 1970), (Pegram and McKenzie, 1991) and (Hipel and others, 1977). The statistical properties of stochastically generated sequences are such that they are considered to originate from the same population and to be generated from the same natural processes that characterize the natural or naturalized sequences on which they are based. The stochastic sequences referred to in this chapter primarily relate to streamflow and are used to study the probabilistic behaviour of yield from reservoirs. However, the same principles for selection and processing can be applied to sequences of rainfall and other hydrological variables of importance to the investigation of water resource systems.
4.2.2.7 Target draft

Target draft is the volume of water that one aims to draw from a reservoir or water resource system to supply requirements over a specified period, generally expressed as an annual total.

4.2.3 Estimation of storage-yield relationships

Many computer models have been developed for the calculation of storage-yield relationships and are relatively easily accessible. This section briefly describes the basic principles underlying these models; further details are provided in 4.2.4 and 4.2.5.

4.2.3.1 Numerical procedure

In its most basic form, yield analysis is a simple sequential mass balance exercise between water entering a reservoir (streamflow, precipitation) and water released or lost from the reservoir (abstraction, evaporation, spillage). The equation to be solved is the following:

\[ S_i = S_{i-1} + I_i + P_i - E_i - D_i - O_i = S_{i-1} + \Delta S_i \]  

(4.1)

where \( S_i \) represents the storage at the end of time interval \( i \), \( S_{i-1} \) represents the storage at the beginning of time interval \( i \), \( I_i \) is the inflow during interval \( i \), \( P_i \) is the precipitation during interval \( i \), \( E_i \) is the evaporation during interval \( i \), \( D_i \) is the draft or abstraction during interval \( i \), \( O_i \) is the outflow or spillage during interval \( i \) and \( \Delta S_i \) is the change in storage during interval \( i \).

Where a time step of a week or longer is used, the average surface area of the reservoir between time intervals \( t_{i-1} \) and \( t_i \) is used to calculate the volumes of precipitation and evaporation.

Where the storage needs to be determined to maintain a certain draft, the equation is solved for different assumed maximum storage capacities (\( S \)) to find, in an iterative way, the capacity where the reservoir is drawn down to barely touching empty, or the minimum operating level. Where a dam already exists or the storage is fixed, the abstraction rate which can be maintained can be determined by substituting storage with draft as the variable in the equation. The sequence of levels of reservoir storage which result from solving the equation is referred to as the storage trajectory.

The trajectory will generally be bounded by the full and minimum operating level states. In general, the trajectory for a given inflow sequence and abstraction rate will be a function of the starting storage level and will differ from starting level to starting level. However, once corresponding full or minimum operating level states have been reached for the range of starting storages, the trajectories from that point onwards will be indistinguishable for a given inflow sequence and abstraction rate.

The period of maximum drawdown of a reservoir, that is, from a full state of storage down to the minimum operating level and recovering until it reaches the full level again, is referred to as the critical period. To reach stability in the analyses, it is important that the critical period be clearly defined by the reservoir trajectory.

Careful inspection of the trajectory with respect to the occurrence of low flow periods remains important, however. A potentially more severe low flow period than defined by the critical period may occur at the beginning or end of the inflow sequence, but where the first or last part of such a potentially more severe low flow period may be truncated by the record length of the inflow sequence available. Should this be suspected, an adjustment may judiciously be made to the abstraction rate by accounting for the net change of storage over the period of the sequence analysed.

In the simplified case described above, the yield of the system was assumed to be equal to the abstraction rate. One may, however, aim to abstract more or less water from a resource rather than the yield of the reservoir or water resource system. The relevance of target draft to the yield characteristics of a water resource system is described in more detail in 4.2.4.

4.2.3.2 Graphical approach

The graphical approach offers a simple alternative for visually presenting the results of sequentially solving equation 4.1.

In a reservoir subject to an inflow \( I \) and draft \( D \), the storage \( S \) at time \( t \) is mathematically defined as:

\[ S_t = S_0 + \int_0^t (I - D) \, dt = S_0 + \int_0^t I \, dt - \int_0^t D \, dt \]  

(4.2)

(For ease of demonstration, the influences of evaporation and precipitation are not included above,
and the draft is representative of all outflow. Spillage would occur where the inflow mass curve exceeds the draft mass curve as shown in Figure II.4.4.)

Plots of the cumulative sums $I^*$ and $D^*$ represent the inflow and draft mass curves, respectively, with $S_0$ being the initial reservoir storage. Figure II.4.4 illustrates how the required storage capacity $S$ is determined for a constant draft $D$ with the constraint that no failure is allowed during the sequence analysed. The procedure employs the concept of a semi-infinite (bottomless) reservoir. The constant draft corresponds to a constant slope of the draft mass curve $D^*$. A line, parallel to $D^*$, is drawn through each peak on the inflow mass curve $I^*$. The design storage capacity $S$ is the maximum vertical distance between any point on $I^*$ and any of the lines that are parallel to $D^*$.

The graphical approach was widely used in the past. However, computing power has increased enormously over the years, facilitating the solution to equation 4.1. In addition, the digital approach offers great flexibility in analysing various scenarios; as a result, the graphical approach is now seldom used, if ever.

4.2.3.3 Influence of record length

Although there are no formal guidelines for the minimum period of record, reasonable stability with respect to yield analyses is generally reached with a record length of 10 to 20 times the critical period. Where little variability in streamflow occurs and where the need is mainly for seasonal storage (less than one year), a minimum record period of 10 to 20 years may be acceptable. However, in semi-arid to arid areas, over-year storage is generally required, as critical periods of 5 to 10 years and longer are common. A record length of 50 to 100 years should preferably be used in such cases.

Even where reasonably long streamflow records exist, worse floods and worse droughts than those historically observed are bound to occur in future. It is also virtually certain that the exact configuration of a streamflow sequence, as recorded in the past, will never be exactly repeated in future. It is evident, however, that the longer the period of record on which the inflow sequence is based, the more reliable the estimation of the yield is likely to be. While historical records are the only factual information available, improved perspective on possible future extreme events can be gained through the stochastic generation of streamflow, as described in 4.2.2.6.

4.2.4 Classifications of yield

The yield characteristics of water resource systems are more complex than can be described by a single formula such as equation 4.1 and requires a more complete description than that already been provided.

Concepts were developed for the classification of yield from a reservoir or water resource system as base yield, firm yield, secondary yield, non-firm yield and average yield (Basson and others, 1988). These facilitate a graphic representation of the behaviour of a reservoir or water resource system as shown in simplified form in Figure II.4.5. The values for defining the diagram are obtained from solving equation 4.1 for various target drafts and by recording the relevant results.

Such diagrams enhance further understanding of the behaviour of a system under various operational conditions. They are particularly useful where water resources are highly utilized, where high variability of streamflow occurs and where yield determination and management of complex water resource systems is necessary.

Base yield is defined as the minimum yield over a specified number of consecutive time intervals that can be abstracted from a river or reservoir system fed by a given inflow sequence while attempting to satisfy a given target draft associated with a specified demand pattern for water and a specified operating policy. The base yield initially increases with increased target draft until a stage is reached when the reservoir is unable to yield continuously at the target draft, resulting in base yields lower than the target draft.
Firm yield is defined as the maximum base yield. For analyses based on historically observed streamflows, reference is made to the historic firm yield to distinguish it from the firm yields derived by probabilistic methods as described in 4.2.5. The historic firm yield associated with a particular reservoir capacity may vary with length of inflow sequence. In fact, it is likely to be smaller for a longer inflow sequence because there is an increased probability of a more severe low-flow sub-sequence occurring.

The yields obtained according to the methodologies described in 4.2.3.1 and 4.2.3.2 are analogous to the historic firm yield.

Secondary yield is the yield that can be abstracted in excess of target draft. As defined, it is withdrawn from a reservoir only while the reservoir is at its full supply level. The assessment of secondary yield for various maximum abstraction capacities can therefore provide a valuable measure of the potential for further development of a water resource system. Secondary yield is often used for the generation of additional (secondary) hydropower or other interim beneficial uses, where appropriate facilities exist.

Non-firm yield is the average yield that can be abstracted from a water resource system in excess of base yield, but not exceeding the target draft. This is not a continuous yield and cannot, therefore, be relied on at a specific assurance of supply.

Average yield is the sum of the base yield and non-firm yield averaged over the period analysed. It provides a measure of the yield which can, on average, be abstracted from a system, but where part of the yield cannot be continuously supported.

Total yield is simply the sum of the base, secondary and non-firm yields.

Further interpretation of the draft–yield response diagram is given by Basson (Basson and others, 1994).

4.2.5 Probabilistic approach

4.2.5.1 Storage–flow dependability relationship

The design and operation of a storage reservoir is an important component of most water resources development projects. While the design storage capacity will depend on the demand for water to be met from the reservoir, the major factor affecting this decision will be the available flow in the river at the location of the planned reservoir. It fluctuates from year to year, however, and this variability must be taken into account.

A correct estimation of reservoir capacity is very important. If it is not sufficient, the project will not serve the community to its full sustainable potential and may lead to wasting scarce water resources. However, over-estimation of the storage capacity may result in the reservoir rarely filling despite its high construction cost, thus rendering the project uneconomical. Therefore, the criteria for choosing reservoir size should include not only the overall demand, but also the reliability with which that demand should be met. For example, 75 per cent dependable yield means that the quantity of water required for irrigation will be availability for at least three out of each four years. A 100 per cent dependable yield means that the required supply of water will be available every year – a 100 per cent success rate – but this can only be achieved if the supply rate is less than that for 75 per cent dependable yield. This in turn would only satisfy a far lower demand.

Different countries have different criteria for planning water resources projects. The concept of a percentage dependability, where a certain level of
failure is acceptable, is frequently adopted in developing countries because they give paramount importance to the economic feasibility of projects. On the other hand, in developed countries – the United States of America, for example – the principle criterion is to meet the requirements for a particular purpose with nearly 100 per cent certainty. The percentage also varies according to the type of services to be provided by the reservoir. As a rule, it may be set at 75 per cent for irrigation, 90 per cent for hydroelectric power generation and 100 per cent for domestic water supply projects.

The following methodology can be employed for determining flow of a certain reliability at a particular point in a river:

(a) Annual gross yield, or the natural flow volume, is also known as the virgin, or historical, flow. It is defined as the flow that would have occurred at that point of the river had there been neither any abstractions from nor additions to the flow upstream from sources outside the river system. In this, both natural seepage and evaporation are ignored. The natural flow can be determined by adding the observed flow, upstream water used for irrigation, domestic and industrial uses both from surface and groundwater sources, increases in water volumes held by the reservoirs (both surface and subsurface) and evaporation losses from the reservoirs, and deducting return flows from different uses from surface and groundwater sources. This is represented by the following equation:

\[
R_n = R_o + R_i + R_d + R_gw - R_{ri} - R_{rd} - R_{rg} + S + E \tag{4.3}
\]

where \(R_n\) is the natural flow, \(R_o\) is the observed flow, \(R_i\) is the withdrawal for irrigation, \(R_d\) is the withdrawal for domestic, industrial and other requirements, \(R_{gw}\) is the groundwater withdrawal, \(R_{ri}\) is the return flow from irrigated areas, \(R_{rd}\) is the return flow from domestic, industrial and other withdrawals, \(R_{rg}\) is the return flow from groundwater withdrawal, \(S\) is the increase in storage of the reservoirs in the basin and \(E\) is the net evaporation from the reservoirs.

If inter-basin transfers are involved – whether into or from the river basin – the amounts thereof will have to be respectively deducted from or added to \(R_i\).

(b) To ascertain the percentage dependability of the flow at a given point on the stream where a continuous record of natural flows for a number of \(N\) years is available, the annual values of natural flows are arranged in a descending order. Each year’s flow so arranged is assigned the serial number from top to bottom and if \(M\) is the serial number of the flow in any year, the percentage dependability for the flow of that year \((D)\) is calculated by applying the formula \(100M/N\). Some authorities prefer the formula to be expressed as \(100M/(N+1)\);

(c) The year that would represent a particularly desired percentage of dependable flow can be directly ascertained by rearranging the relationship to \(M = DN/100\) or \(D(N+1)/100\) and the amount of flow of that dependability read out from the natural flow series. In cases where the derivative of \(M\) is not a whole number, a small adjustment may be required in the values of flows of the two years between which \(M\) falls so as to achieve the closest dependable flow corresponding to the exact percentage of dependability;

(d) The same results are obtained by creating an ascending series of natural flows rather than a descending series.

The natural flows worked out by equation 4.3 can also be used to apportion the flow in river among various potential users such as riparian States.

### 4.2.5.2 Risk of failure and reliability of supply

Many definitions of failure of a water resource system can be formulated. The definition favoured in this chapter is where failure of a water resource system is defined as the inability of the system to supply the base yield associated with a specific target draft. Risk of failure of a water resource system can be defined as the probability of not being able to supply the base yield associated with a specific target draft at least once over a specified time horizon.

It is common practice to make use of the recurrence interval concept to quantify risk of failure of a water resource system. Typical recurrence intervals associated with large systems are 1 in 20, 1 in 50, 1 in 100 and 1 in 200 years. The probability of failing in a particular year, the annual risk of failure, is the reciprocal of the recurrence interval. Therefore, a 2 per cent probability of failure in any one year is equivalent to a recurrence interval of 50 years. Thus:

\[
R = 1/T \tag{4.4}
\]

where \(R\) indicates the annual risk of failure and \(T\) indicates the recurrence interval of failure.

The probability of successfully meeting the requirements for water in a particular year, the annual probability, is simply one minus the annual risk of
Annual reliability of supply is therefore related to the recurrence interval of failure by the following relationship:

\[ r = 1 - 1/T \]  
(4.5)

where \( r \) is the annual reliability of supply.

The long-term risk of failure is related to annual risk of failure by the Bernoulli probability relationship:

\[ R_n = 1 - (1 - R)^n = 1 - (1 - 1/T)^n \]  
(4.6)

where \( R_n \) is a long-term risk of failure and \( n \) represents a planning period (length of sequence) in years.

Although some assessment of the risk of failure may be made from analyses of an historical streamflow sequence, the confidence that can be attached thereto is normally not very high, unless exceptionally long records exist. Stochastically generated streamflow sequences are therefore employed as a means of increasing the sample size of possible configuration of streamflow sequences in order to obtain improved statistical assessment.

4.2.5.3 Draft–yield reliability characteristics

Through the analyses of a large number of stochastically generated streamflow sequences, typically between 200 and 2,000 sequences of the same duration as the historic sequence, a probabilistic assessment of the characteristic behaviour of a water resource system can be obtained.

While many well-proven stochastic hydrological models have been developed, extensive tests and re-sampling need to be performed to ensure that the basic parameters of the historical records, on which the models are calibrated, are well preserved for each point of interest. The validation of models is particularly important in semi-arid and arid areas where large variations in streamflow occur.

Figure II.4.6 reflects the addition of probabilistic information on the basic draft–yield response diagram. The partial box plots indicate the sampled distribution of base yields resulting from the analysis of a large number of generated stochastic sequences, at target drafts of 70, 80 and 90 million m\(^3\) per year. The shape and somewhat steeper decline of the historic base yield line, compared with the 1:100 probabilistic base yield line, are attributable to the specific configuration of the historic critical period, whereas the probabilistic line displays a combined value from the analysis of a large number of inflow sequences.

Additional perspective can be gained by presenting the draft–yield reliability characteristics as shown in Figure II.4.7. These curves also form the basis for the assessment and management of complex multi-reservoir systems as described in the sections to follow.

For the example in Figure II.4.7, should a yield of 60 million m\(^3\) per year be required from a system at a risk of failure not exceeding 1/100 years, it can be achieved by imposing a target draft of
82 million m³ per year on the system (see dotted line). The additional 22 million m³ per year will then be available at a risk of about 1/80 years. The additional water can be used for applications in which a lower assurance of supply is required, such as the generation of secondary hydropower or the support of adjoining or other water resource systems. Alternatively, the storage can be reduced so that a firm yield of 60 million m³ per year can be obtained at the specified 1/100 year risk of failure.

4.2.5.4 Short-term yield characteristics

Whereas the long-term yield-reliability curves capture the long-term yield capabilities of a water resource system and provide perspective on the long-term average behaviour thereof, they do not contain sufficient information to make short-term operational decisions. There the influence of the ruling state of storage is of paramount importance. However, decisions with respect to real-time water allocations cannot be based solely on the current situation, but should also account for safeguarding the supply for some period into the future. The duration of this safeguarding period should be a few time steps longer than the time step between major operational decisions.

Short-term draft–yield reliability characteristics are developed in the same way as the long-term family of curves, except that short-term curves also relate to a specific starting storage. Curves therefore need to be developed for a range of starting storages. Because of the shorter duration of the sequences used, typically two to five years, many sequences may not span a critical period. Therefore, significantly more short-term sequences need to be analysed to achieve convergence than for the long-term analyses. More detail on the practical application of short-term characteristic curves in the real-time operation of water resources systems is given by Basson and others (1994).

4.2.5.5 Reservoir filling times

When a new dam is built, a certain stage of storage in the reservoir must be reached before the full yield can reliably be abstracted from it. In semi-arid and arid areas, as well as where water resources development has practically reached its full potential, it may take several years after the start of impoundment to reach the first filling of the reservoir, even if no water is abstracted during this period. This can have a major impact on the planned development phasing, as well as on the economic feasibility of a project.

Probabilistic projections of filling times for new reservoirs can be obtained by determining the reservoir trajectories for a large number of stochastic inflow sequences, starting empty. A practical duration for the analyses should be selected, while various options for incrementally imposing draft on the reservoir may also be tested. Figure II.4.8 shows a probabilistic assessment of the filling time for a reservoir.

4.2.6 Multi-purpose reservoirs and operating rules

Most storage reservoirs serve a number of purposes, as shown in 4.1. It is generally not practicable to allocate a fixed portion of storage for each purpose. In most cases, such an allocation is restricted to emergency purposes. For example, a buffer zone is often created immediately above the dead-storage zone and is reserved for use in exceptional circumstances, such as flushing the downstream river section in case of accidental contamination, emergency water supply to deal with sudden health hazards or fire fighting. However, most purposes are served from the same storage and their requirements are accommodated by complex release rules for reservoir operations. Different users will require different quantities of water at different times, with different assurances of supply.

Reservoir releases are often formulated in terms of rule curves that indicate the rate of release as a function of the ruling or instantaneous storage and the time of the year. Different assurance of supply requirements also imply that different categories of users have different tolerances to cope
with some degree of failure in their water supply. In many parts of the world it is often not feasible for a project to be developed to meet the full requirements for water all the time. Strategies are then introduced to incrementally curtail the supply of water to some users when critical levels of storage are reached.

Some uses of a reservoir do not require the release or abstraction of water. However, certain storage limits at specific times of the year, which may impact on the yield characteristics of the reservoir or water resource system, should be observed, for example, when reservoirs are used for flood control, recreation, salinity control and when environmental considerations are involved.

The design and operation of multi-purpose reservoirs require complex analyses, which are generally carried out by iterative methods that involve adjustments of the rule curves and evaluation of the effects on all individual purposes in order to optimize water resource system management. Formal optimization techniques can also be employed to find the best solution among certain trade-offs. It is important that all potential uses and users of a reservoir be taken into account during the planning stages of a project and that consideration be given to the real-time operating rules in advance of the construction. Where multi-purpose reservoirs or complex water resource systems are involved (see 4.2.7), it is advisable that the operating rules be developed as part of the planning process.

Examples of rule curves and approaches to real-time operating rules can be found in Box and Jenkins, 1970; Basson and others, 1994; Loucks and others, 1981 and Svanidze, 1977.

4.2.7 Multi-reservoir water resource systems

Owing to the high degree of water resources development in many parts of the world and its steady growth in others, the occurrence of multiple reservoirs in a basin is becoming more and more common. These may be in a series downstream of one another on the same river, in parallel on branches of the river, or various combinations thereof in a catchment or river basin. Reservoirs in adjoining catchments or different basins may also be linked together through the transfer of water, resulting in even larger and more complex water resource systems, such as shown in Figure II.4.9.

Where two or more reservoirs are linked together through their location on the same river system or via transfers, one will impact on the other, even if only with respect to shared downstream release requirements. Such reservoirs are inherently part of the same system, and need to be recognized as such in the management of the resource. In many instances, the introduction of inter-reservoir operating rules may be required. Any new additions also need to be evaluated and later managed in the context of the overall system.

Where initial single reservoir projects develop into multi-reservoir systems over time, the operation of existing components may have to be changed. Major changes are often difficult to implement because of many legal, political, economic and physical constraints. Accordingly, the level of optimization that can be reached in practice in such cases is generally low.

Where greater flexibility exists or can be added, significant benefits may be achieved from the operation of reservoirs as one interconnected system. Generally, this is further enhanced where different basins are linked together through the transfer of water. Individual reservoirs or sub-components of a system may, for example, be operated at a target draft which is in excess of the firm yield of the reservoir or subsystem, but with the knowledge that it can be supported from other parts of the system during periods of deficient yield. In this way an overall yield can be obtained which is greater than the sum total of the firm yields of the component parts of the system.

It is strongly recommended to add a probabilistic dimension to the management of multi-reservoir water resource systems. This requires that stochastic streamflow sequences be generated for each point of interest in the system. Of specific importance in this regard is that the cross-correlation among observed streamflow sequences at the respective points be carefully preserved. Much of the confidence related to the probabilistic management of water resource systems is dependent on the accurate replication of these characteristics in the generated sequences.

The multi-dimensional problem presented by the probabilistic management of water resources as one interconnected system is comprehensively covered in the literature. It is evident that determination of the yield characteristics, as well as operational management of multi-reservoir water resource systems, can be very complex and can generally be done solely with the aid of sophisticated computer
models. Several models have been developed, most of which can be obtained from the relevant organizations or institutions, normally subject to some form of licence or agreement. Reference may be made to Hatch Energy, Canada (www.hatchenergy.com); BKS Group (www.bks.co.za) and Department of Water Affairs and Forestry, South Africa (www.dwaf.gov.za); Danish Hydrological Institute, Denmark (www.dhisoftware.com); Hydrological Engineering Centre, US Army Corps of Engineers (www.hec.usace.army.mil) and Deltares, Netherlands (www.wldelft.nl).

4.2.8 Incidental effects of reservoirs

The purpose of this section is to create a general awareness of the incidental effects of reservoirs; however, the subject is not addressed in great detail. The section focuses on the effect of impoundment in reservoirs created by dams, not with the direct impacts of dam structures or hydropower stations, such as the creation of barriers to fish migration. The important social impacts of reservoirs are not addressed either.

4.2.8.1 Effects on hydraulic and hydrological regimes

The construction of a dam causes changes in the hydraulic and hydrological regimes downstream. Consumptive uses of water reduce the mean flow, while reservoir regulation changes the seasonal distribution of flow and generally reduces its variability. The detention of water in the reservoir causes sedimentation and results in released water with greater transport capacity than the inflow, which can cause erosion below the reservoir. The decrease in hydraulic gradient may cause backwater and sedimentation problems in the river channel upstream of the reservoir.

4.2.8.2 Environmental effects

Environmental effects are of increasing concern in the planning and management of water resources projects.

The construction of reservoirs generally has a very important impact on the environment. Where the
storage volume is large in relation to annual runoff and there is a high water nutrient load level, eutrophication can have a significant impact on the quality of the water as a result of long residence times. The reservoir has a major effect on the temperature and oxygen content of the release water. Less turbid water resulting from sediment deposition allows for deeper light penetration which may cause algal blooms. The regulation of flow is also associated with a change in the nature of land use and increased water use downstream of the reservoir. This generally results in an increase in the amount of wastewater produced, which may lower the quality of water in the receiving river.

Changes of this nature are a major concern. However, reservoirs also cause changes with beneficial effects. In many cases, if managed appropriately, the environment in the vicinity of reservoirs and downstream may be greatly improved by providing recreational, aesthetic, ecological and health benefits.

It is of primary importance to provide monitoring facilities for measuring environmental factors both before and after construction and to assess continuously all environmental effects of storage reservoirs.

4.2.8.3 Environmental flow requirements

As a means of mitigating the impacts of reservoirs on downstream aquatic life, releases are made to at least partly recreate some characteristics of the natural flow regime necessary for maintaining healthy ecosystems. Such releases need to be allowed for during the planning phases of a project and in determining the yield characteristics of a water resource system. Environmental flow requirements can have a major impact on the abstractable yield from a system, particularly where a high conservation status of a river needs to be maintained. Several cases have also been recorded where, because of the growing awareness and appreciation of environmental issues, allowable abstractions had to be substantially reduced in favour of larger environmental releases.

The determination of environmental water requirements is a specialist field of its own, and is beyond the scope of this chapter (see Volume I, Chapter 7, and Chapter 3 of the present volume). Methods have, however, been developed which can be used by water resource practitioners to obtain an approximation of environmental water requirements for initial planning purposes (Hughes and Hannart, 2003).

4.2.8.4 Other effects

Backwater effects produced by the impoundment in reservoirs, as well as fluctuations in reservoir level, such as may be caused by flood flows, wind set-up, wave action and periodic undulations of the water surface (seiches), may be reflected in short-term variations in local water balance calculations. However, these effects relate mainly to design aspects and the safety of the dam structure, as well as the safety of people and developments in the immediate proximity of the reservoir basin, and are therefore not considered further in this section.

4.2.9 Remote-sensing estimates of reservoir capacity

The delineation of surface water bodies and the inventory of surface water supplies, including lakes, ponds and reservoirs, have historically been developed on the basis of maps and photo interpretation techniques, but digital multi-spectral data have recently been used as well. These data can be subjected to automated analysis so as to achieve repetitive, rapid results that in many cases also meet accuracy requirements. In general, the accuracy of detecting and measuring water bodies has been largely a function of proper identification of water and sensor spatial resolution. Identification problems involve confusion with areas with similar appearance, such as cloud shadows, dark soils and urban areas. However, aerial photograph interpretation can be used to minimize these errors and check the initial results. Therefore, for extremely accurate work, aerial photographs still provide the best data sources. Satellite data also provide good sources for determining morphometric parameters, such as length, width and surface water area for different elevations, if the resolution is suitable for the specific use.

From a remote-sensing perspective, water has a relatively low reflectance, especially in both the near-infrared and visible portions of the electromagnetic spectrum. This will help to separate urban areas, fields and sometimes cloud shadows, and ambiguities produced by variations in atmospheric transmission (Engman and Gurney, 1991). The use of Thematic Mapper data with its 30-m nominal resolution will increase these accuracies. Furthermore, data from the SPOT satellite system are expected to yield improved accuracy.

Remote-sensing, for the most part, can only determine the surface of the water and cannot measure the volume directly. Mapping surface water area in
4.2.10 Climate change

There is growing evidence that global temperatures are rising and that the rate of increase may be substantially higher than has occurred in the past (IPCC, 2001). Some global circulation models suggest that this could cause changes in annual precipitation and increase the variability of climate in certain regions. Scenario analyses for assessing the potential impacts of climate change on streamflow indicate that in some areas streamflow could decrease by as much as 10 per cent by the year 2015 (Schulze and Perks, 2000).

Such changes in climate would have significant impacts, not only on the yield characteristics of water resource systems, but also on the requirements for water to be abstracted from the systems. It is wise, therefore, to anticipate the possibility of climate change and perform scenario analyses for areas that could be vulnerable so as to assess the potential impacts that climate change might bring. Although it is appropriate that the potential impacts of climate change be considered in the long-term planning of water resources systems, a balance should be sought between preparedness and possible overreaction to prevent valuable resources from being wasted.

Sensitivity analyses, focused on how the yield characteristics of water resource systems, could be affected by climate change, can be performed by incrementally changing the mean and/or standard deviation with respect to the synthetic generation of streamflow. Indications of what may be regarded as a realistic extent of such changes can be derived from scenario analyses with the aid of global circulation models, but will probably largely remain subject to personal judgement.

4.3 Flood management

4.3.1 General
A flood is a “rise, generally brief, in the water levels in a stream to a peak from which the water level receded at a slower rate” (UNESCO/WMO, 1992). Some floods overflow the normal confines of a stream or other body of water and cause flooding over areas which are not normally submerged. Floods, high or low, are part of the natural hydrological cycle and are generally an outcome of a complex interaction between natural random processes in the form of precipitation and temperatures with the basin or watershed characteristics. In general, the magnitude of a flood depends on the following factors:

(a) Volume, spatial distribution, intensity and duration of rainfall and snowmelt over the catchment;
(b) Catchment and weather conditions prior to the rainfall event;
(c) Ground conditions such as land use, topography and so forth;
(d) The capacity of the watercourse to convey the runoff (including that due to ice jams or log jams);
(e) Impact of tidal or storm surges.

Flood plains offer many advantages for human settlement and socio-economic development because of their proximity to rivers that provide rich soils, abundant water supplies and a means of transport. Floods also replenish wetlands, recharge groundwater, and support fisheries and agricultural systems, thereby adding to the attractiveness of flood plains for human settlement and economic activities. At the same time, flood hazards produce the most adverse impacts on the economy and safety of people. Floods continue to lead all natural disasters in terms of the number of people affected and resultant economic losses (Munich Re, 2006). The struggle of humankind against this natural hazard is as old as the history of human settlement. Over recent decades, this struggle has seen a gradual shift from flood control to flood management. This chapter provides an overview of efforts that can be made to mitigate the adverse impacts of floods while making use of the flood plains.

4.3.2 Flood management strategies
Flood control refers to the specific process of providing and operating structures designed to eliminate or minimize the damaging effects of floods by...
detaining, constraining or diverting flood flows up to an economically based design limit (ICID, 1996; Framji and Garg, 1978). However, flood management refers to the overall process of preventing and mitigating the extent of flooding and reducing the flood risks in a holistic manner. Flood risks, which can be defined as the expected losses from flood events spread over a specified time period, are a construct of the following factors:
(a) Magnitude of the flood hazard expressed in terms of frequency and severity (depths of inundation and related velocities);
(b) Exposure of human activities to flooding;
(c) Vulnerability of the elements at risk.

Providing absolute protection to flood-prone areas for all magnitudes of floods is neither possible from a practical point of view nor economically viable. Hence, a practical approach to flood management would be to provide a reasonable degree of protection against flood risks at an acceptable economic cost through a combination of structural and non-structural measures. Over the years, flood protection measures have played an important role in safeguarding both people and socio-economic development from flooding. However, during the last decade or so, these measures have been complemented with non-structural measures such as flood forecasting and land-use regulation in response to a perceived need for a paradigm shift from flood control to flood management.

There are four major flood management strategies aimed at reducing flood risks:
(a) Modifying flood characteristics;
(b) Changing society's susceptibility to flood damage;
(c) Reducing the loss burden per capita;
(d) Bearing the loss.

Flood modification methods aim at changing the volume of runoff, the time taken to attain the peak, the duration of the flood, the extent of the area susceptible to flooding, the velocity and depth of flood waters and/or the amounts of sediments and pollutants carried by the floods. These methods involve flood protection by means of physical controls such as dams and reservoirs, levees and embankments, channel modification and flow diversion and catchment treatment.

Measures can be taken that reduce the susceptibility of economic activities to damage, with certain activities focused on the flood plain. These include land-use regulation, flood-proofing, flood forecasting and flood warning.

Reducing the loss burden consists of actions designed to modify the incidence of the losses per capita, either by spreading them over a larger segment of the community than that which is immediately affected or spreading them more evenly over time. This is a strategy for reducing the losses by means of actions planned to assist the individuals and the community in the preparatory, survival and recovery phase of floods, such as emergency preparedness, evacuation, flood fighting, post-flood recovery and insurance programmes (these measures are complementary to those discussed in the previous two items).

Bearing the loss denotes living with floods. With the growing emphasis on considering the whole range of responses to flood hazards in cost-benefit terms, bearing the loss can often be considered the most acceptable solution.

The development of policies, strategies and plans to combat the risks associated with flooding or any natural hazard should be based on a comprehensive assessment of the risks involved. This requires an integrated approach whereby a wide range of flood management measures should be considered. It is necessary to look at the overall situation, compare the available options and select a strategy that is most appropriate to a particular situation. While recognizing the pros and cons of various structural and non-structural measures, a good combination of both types of measures needs to be evaluated, adopted and implemented. For example, a levee in one part of town may be positively supplemented by land-use adjustments in an unprotected floodway area and by structural adjustments in a sparsely built-up sector, or flood control by using reservoirs may be combined with land-use regulations.

4.3.3 Integrated flood management
Traditionally, flood management has focused on defensive practices. It is widely recognized, however, that there is need for a shift from defensive action to the proactive management of risks. Integrated flood management, designed to integrate land and water resources development in a river basin within the context of integrated water resources management, seeks to manage floods in such a way as to maximize the net benefits from flood plains while minimizing the loss of life from flooding (WMO, APFM, 2004). Thus, occasional flood losses can be accepted in favour of a long-term increase in the efficient use of flood-prone areas. There are five objectives in integrated flood management:
(a) Manage the water cycle in so far as it relates to land, as a whole;
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(b) Integrate land and water management;
(c) Adopt the best mix of strategies;
(d) Ensure a participatory approach;
(e) Adopt integrated hazard management approaches.

For a detailed discussion of these objectives, please refer to WMO (APFM), 2004.

For flood management to be carried out within the context of integrated water resources management, river basins should be considered integrated systems. The multi-faceted measures include, for example, socio-economic activities, land-use patterns, hydromorphological processes, public awareness, education, communication and stakeholder involvement with transparent decision-making. These need to be recognized as constituent parts of these systems that are duly embedded in non-structural measures.

Flood management is an interdisciplinary pursuit involving different sectors of the economy and the various departments and institutions which have an impact on the magnitude of floods, as well as the implementation of flood management measures. For this, the linkages between various relevant sectors become very important, and the most important key is cooperation and coordination across institutional boundaries. While the mandates of many institutions may cover only part of the river basin or one sector, others extend well beyond the basin boundary. Effective communication across institutional and disciplinary boundaries is at the core of integration, which can take place only if there is a clear understanding of common goals. Emphasis, therefore, should be placed on the adoption of flexible strategies suited to each flood-prone region, which is characterized by various physical, social, cultural and economic situations. Further, it is important to evaluate the different options and their relative advantages and disadvantages.

Loss of life can be avoided if reasonably accurate and reliable forecasts are provided to flood-plain occupants in a timely manner. However, this has to be supported with adequate preparedness measures and response mechanisms designed to vacate people from the threatened zones. Flood hazard maps, also referred to as flood maps, flood risk maps or flood-plain zoning maps, show areas likely to be flooded with a given probability and provide a long-term advance warning that serves as a basis for helping people to make their own decisions as to whether and where to live and invest in the flood plain (WMO, 2006a). These tools play an important role in building awareness among various stakeholders of the risks of flooding and help them organize flood response activities. Flood-plain zoning, which can be one further step, can be of great value, but it also has limitations because of the difficulties of enforcing the related rules and regulations – particularly in developing economies with population pressure.

4.3.4 Structural measures

4.3.4.1 Design floods

A design flood is defined as the flood hydrograph or the instantaneous peak discharge adopted for the design of a hydraulic structure or river control after accounting for political, social, economic and hydrological factors. It is the maximum flood against which the project is protected and its selection involves the selection of safety criteria and the estimation of the flood magnitude that meets these criteria. This subject is covered in detail in 5.10.

Although a flood control structure is installed to control future floods, its design is generally based on analyses of past floods. Such an extrapolation of the past hydrological series for the future may not always be appropriate, however, owing to the change in meteorological events or in the change in the hydrological response of the basin. Anthropogenic influences due to the growth of population and higher standards of living may result in intensified land development. The extension and intensification of urbanization often contribute to increased volumes and peak flows of surface runoff and sediment transport. Deforestation may result in an increase in sediment yield and destabilization of river morphology. Forests may be transformed into agricultural lands and the drainage of agricultural lands improved. An assessment of the hydrological effects of upstream changes can be made by using deterministic hydrological models (see Chapter 6) to evaluate their impact on the risk of downstream floods.

A number of studies on the potential impacts of climate change on flooding have been carried out as part of the work of the Intergovernmental Panel on Climate Change (IPCC, 2007). At this time, it is not possible to predict potential increases in flood peaks due to climate change for specific basins with the degree of certainty necessary for their incorporation into the planning and design process. However, adaptive management techniques, such as revision of criteria for determining the freeboard on levees and other works or judiciously modified
operating procedures for control structures, hold the promise of accommodating the potential increase of extremes caused by climate change.

4.3.4.2 Flood retention reservoirs

The over-bank spilling of a river resulting in flooding does not generally occur for long periods, even during the flood season. High-magnitude floods are caused by severe storms associated with extreme meteorological systems such as cyclones and intense monsoons, and subside within a reasonable period. Depending on the catchment characteristics and storm track, flood discharge fluctuation due to heavy precipitation, followed by a relatively drier spell, can be used to advantage to moderate the flood through a variety of storages during high discharge. Storage is generally provided behind a dam on an upstream reach of the river, but distributed storage basins on the flood plains can also be used.

4.3.4.2.1 Flood storage capacity of reservoirs

The volume of storage that must be provided to retain flood waters in a reservoir depends on the following factors:
(a) Volume, peak flow, duration and other characteristics of the upstream flood that is to be moderated;
(b) Storage requirements to meet various other water demands;
(c) Carrying capacity of the downstream channel;
(d) Extent of flood moderation required.

If floods are highly variable over the year and occur only during a certain season, the reservoir may have to play a multi-purpose role to meet various water demands in addition to flood moderation. If so, reservoir capacity will be fixed mainly on the basis of other water demands, with only a certain storage reserved specifically for flood moderation during the flood season. In such cases reservoir levels are drawn down before the flood season begins and are refilled as the season passes. Flood storage can be provided either in on-stream or off-stream reservoirs. If such dedicated storage cannot be provided, some flood abatement can be achieved through the use of carefully designed reservoir operation schedules.

The multiple uses of reservoirs are also considered under 4.2.6.

4.3.4.2.2 Design considerations

Flood abatement is achieved by detaining and later releasing a portion of the peak flood flow. The amount of storage required, or detention storage, is generally specified as that part of the reservoir storage that can produce a given reduction in the flood peak of a given magnitude or of a given probability of occurrence. The following basic types of storage can be distinguished:
(a) Regulated storage, either in an on-stream or an off-stream reservoir;
(b) Unregulated storage in an on-stream reservoir;
(c) Unregulated storage in an off-stream reservoir.

The storage capacity needed to achieve a given effect will depend on the type of storage used. The flood-transformation effects of each type of storage aiming for the same flood-peak reduction are shown in Figure II.4.10 and are discussed in the following subsections. In practice, the effect of a flood control reservoir is generally a combination of regulated and unregulated storage.

4.3.4.2.3 Regulated detention storage

Full control over the flood detention storage of a reservoir provides the highest efficiency of flood mitigation because water storage can only begin after the highest permissible flow, also known as the non-damaging flow, has been reached downstream from the reservoir. Therefore, only that portion of floodwater that is apt to cause damage is stored.

Control over storage is achieved by the regulation of gated outlets in the case of an on-stream reservoir and of gated intakes and outlets in the case of an off-stream reservoir. In an on-stream reservoir, full control is achieved only if the outlet has a sufficient capacity to release the non-damaging flow when reservoir storage is at its minimum, and if the release of water from the detention storage can be fully regulated. In an off-stream reservoir, full control is achieved only if the intake has a sufficient capacity to prevent the rise of flow in the downstream section of the river above the non-damaging flow.

![Figure II.4.10. Effects of reservoirs on floods – regulated storage (a), unregulated on-stream storage (b) and unregulated off-stream storage (c)](image-url)
and if the release of the detained water can also be regulated.

The design flood for the determination of the flood detention storage capacity of a reservoir need not to be the same as the one used for the design of its spillway because the dam’s safety requirements generally differ from flood protection requirements downstream from the reservoir.

### 4.3.4.2.4 Unregulated on-stream detention storage

The storage above a fixed spillway crest of an on-stream reservoir is generally regarded as unregulated for design purposes, even though it may be partially regulated by release through gated outlets and turbines. However, for the design of unregulated detention storage, these releases are either considered constant during the passage of a flood, or the outlets are considered closed. The former condition is generally adopted for the assessment of the normal downstream flood control effects of the reservoir, while the latter condition is applied for the assessment of the dam safety.

Unregulated detention storage plays an important role in the safety of a dam against overtopping. Its design is interlinked with the design of the dam spillway and must be based on the same design flood as the spillway itself. Safety considerations in spillway design require that the reservoir be regarded as filled up to the spillway crest at the beginning of the design flood.

As the comparison of parts (a) and (b) of Figure II.4.10 indicates, unregulated storage is less efficient in flood-peak reduction than regulated storage. This is because unregulated storage begins filling even before it is needed.

### 4.3.4.2.5 Unregulated off-stream detention storage

Unregulated off-stream detention storage arises in off-stream reservoirs, sometimes called polders because of their resemblance to real polders. They are constructed by enclosing a part of a flood plain within a dyke whose crest at the upstream end is lowered to form a sill, thus providing an intake to the enclosure. When the river stage at the upstream end rises above the sill crest, the polder starts filling by overflow over the sill. The fact that the river bypasses the reservoir makes the unregulated off-stream storage more efficient than unregulated on-stream storage because unnecessary filling starts later. (See parts (b) and (c) of Figure II.4.10.)

### 4.3.4.2.6 Operation considerations for design purposes

Flood detention storage is frequently provided in multi-purpose on-stream reservoirs with gated outlets with a capacity sufficient to provide a high degree of control of the storage. These reservoirs always have some incidental ungated flood detention storage and, in many cases, part of the gated storage is reserved for flood detention. In addition, storage designated for other uses may occasionally be used to moderate floods. Although this diversity offers flexibility, it makes the flood reduction strongly dependent on the mode of reservoir operation. Therefore, in such cases, it is necessary to analyse many different operation modes during the early stages of design because the results affect the selection of the design variables for the project.

Controlled storage reservoirs are operated by regulation schedules, or set of rule curves, which aim at impounding part of the inflowing flood above a specified safe amount during the rising stage. When the storage capacity is fully used, and depending on the inflow, the outflow is increased to ensure that the design reservoir levels are not exceeded. Thereafter, the impounded flood water is released in such a manner as to empty the reservoir within a reasonable time so that it is available for receiving the next flood, all the while keeping the rate of release as far as possible within the safe limits for the lower reaches. Such an operation should be accompanied by flood warnings to the downstream communities.

In multi-purpose reservoirs, interests such as irrigation, hydropower generation and flood control generally compete with one another even when the reservoir is owned by a single State or agency. This conflict may be heightened when more than one country is involved. Irrigation and hydropower needs generally dictate the filling up of reservoirs as soon as possible and their retention at as high a level as possible. To achieve flood moderation, the reservoir levels should be kept as low as possible and the reservoir depleted as soon as possible after a flood so as to be of use for flood absorption during the next flood event. A reservoir is more effective for flood moderation if, apart from the incidental storage available in reservoir on a river, storage space is allocated for flood detention and is not encroached upon. Multiple uses of reservoir storage therefore imply a compromise, which inevitably results in less than the maximum possible benefits for any one single user but which realize the maximum benefit for the project as a whole. (See 4.2.6.)
The availability of accurate and timely inflow forecasts is essential to optimize the overall benefits derived from a reservoir. In particular, it will be important to forecast the arrival of two or more floods in close succession, with the possibility that the detention storage filled by one flood may not be emptied before the arrival of the next. If such forecasts are missing or inadequate, the effectiveness of multi-purpose operation can be hampered and the ability to moderate the flood will be reduced.

Continued and effective moderation of high floods by reservoirs, over many years, has a tendency to bring a false sense of security among those who live and work in the downstream reaches. This may lead to encroachments on the flood plains and development on the riverbanks and in turn, a reduction in the conveyance of the channel downstream. This can result in serious damage when a large flood has to be let down from the reservoir. Flushing doses, not exceeding the discharge against which protection is given in the downstream valley, should be released in order to keep the river channel alive and deter encroachment in the flood plains. All such releases should be accompanied by an advanced flood warning.

It is therefore essential to formulate and use specific rule curves or preset procedures for the operation of multiple-purpose reservoirs in order to maximize the benefits from the project while ensuring the safety of the structures and downstream communities.

4.3.4.7 Sedimentation effects

The deposition of sediment in a reservoir reduces a reservoir's storage capacity and performance. Reservoir design should allow for allocation of part of its storage capacity for sediment deposition to prevent premature reduction of its active storage capacity. However, designated storage can be inadequate, affecting flood detention storage significantly. The upstream part of the reservoir may be affected first by the sedimentation process. Thus, a reservoir's flood moderation efficiency may decrease with time. This should be considered in long-term flood protection planning so that timely alternatives can be developed and appropriate levels of protection provided for the system. (See 4.2.2.2.)

4.3.4.3 Other structural measures

4.3.4.3.1 Bypass and diversion channels

Diversion of river water may be employed to keep the downstream discharges within the conveyance of a river system. Flow may be diverted all or in part into a natural or artificially constructed channel lying within, or in some cases outside, the river's flood plains. The diversion may move water from one river to another, to a depression or to the sea, or it may be returned to the same river channel some distance downstream. Diversion of floodwater from one river to another involves the following hydrological considerations:

(a) Determination of a design-flood hydrograph for both rivers;
(b) Separation of the part of the flood hydrograph to be diverted;
(c) Flood routing of the diverted flow through the diversion channel;
(d) Combination of the diverted flow with floods which may occur in the receiving river;

Estimation of the revised flood frequencies at the downstream segments of the rivers concerned.

Care should be taken to evaluate the phasing of the superimposed floods in the receiving river as well as the backwater effect that may cause an increase in the flood risk in the reach upstream from the diversion's discharge point in the receiving river.

4.3.4.3.2 Drainage improvement and channel modification

Congestion of surface water drainage due to inadequacy of natural or artificial drainage systems results in flooding in areas with moderate ground slopes. In such cases, effective flood management can be achieved by increasing the capacity of the existing drainage channel or by constructing supplementary channels for accelerated evacuation of floodwater. Similarly, channel modifications generally aim at boosting channel conveyance capacity by deepening and widening the channel, cutting meanders, shortening channel length, clearing vegetation and possibly lining a channel to reduce its resistance to flow. This results in increased flow velocity and lower water levels with flood reduction along the modified reach.

It is important to note, however, that channel modification and drainage improvement cause increases in flood peaks downstream. The effects of such works can best be assessed by hydraulic routing methods (6.3.6) with proper consideration of the interaction between the floods in the main channel and in the tributaries downstream. The possibility of an increase in magnitude and duration of flooding in the downstream area should be considered when planning such schemes through hydraulic modelling of the entire drainage system.
4.3.4.3.3 **Levees and floodwalls**

The oldest, most common and quickly constructed means of flood protection, which is often economical, is a system of levees, also called embankments or dykes. Levees are constructed either on riverbanks in a general direction parallel to the flow of the river or surrounding riparian areas so that they can serve as artificial high riverbanks during high floods and prevent flooding. Levees are constructed mainly from earth and must be resistant to hydrostatic pressure from floods, erosion, piping failure and seepage. Resistance can be achieved by building levees with a broad base. As a result, even moderately high levees occupy a large base area, and in terms of land costs, can be prohibitively costly in urban and industrial locations. In developed areas, where adequate space is not available or land is too expensive for an earthen embankment, concrete or masonry floodwalls may be a more economical, socially acceptable option. River-training works such as spurs, studs and revetments are sometimes necessary in combination with the levees to protect them from flooding. To achieve proper levee design, attention should be paid to the following factors:

(a) Levee alignment;
(b) Design flood levels;
(c) Design freeboards;
(d) Structural design of levees;
(e) Drainage sluice location and design.

The height of a levee system is determined in such a manner as to provide the area concerned with a certain degree of protection defined according to the economic value of the protected area and to local or national decisions as to what is regarded as acceptable risk. For further information on risk, see 4.2.5.2 and 5.10.8. The design is generally set in terms of protection from a design flood with a certain probability of occurrence within specific periods, for example, the 1-, 10-, 25-, 50-, 100- or 1 000-year flood. Design water levels should be calculated on the basis of the hydraulic conditions in the entire basin. On rivers where human activities influence the water regime (upstream reservoirs, levees or barrages), their effect should be considered and, on rivers subject to frequent ice jams or landslides, water levels should be calculated according to the highest backwater levels caused by downstream jams. Construction of high levees tends to be unattractive in view of the cost considerations. Another consideration is the potential damage that results when levees are overtopped. Design water levels in ice-prone rivers should be calculated on the basis of ice-free observations if the flow regime is natural.

Freeboards above the design flood level are added to ensure that design floods do not overtop the levee; uncertainties in design flood calculations, including those due to likely climate change, are accounted for; seepage does not cause significant flow within the body of the levee to cause piping and waves do not spill over the crest of the levee. Depending on wave conditions and the slope of the levee on the waterside, the freeboard should normally be in the range of one to two metres. Freeboards can be provided by building floodwalls on the crest of the levee to reduce costs. The loading of the levees, not only in terms of force, but also in relation to their susceptibility to seepage, depends on the duration of the floods. Thus a statistical study of the duration of certain water levels may help to design and construct seepage-resistant embankments. Drainage sluices, service roads on the crest or on the toe, fuse plugs and toe drains are examples of important components to be considered as part of the design of the levees.

The alignment of the levees and the width of the unprotected flood plains is governed by and influences the upstream and downstream hydraulic conveyance conditions of the channel. The location of the flood levees should consider the effect of the spacing between the embankments on the new water levels upstream due to the loss of valley storage, otherwise available for flood moderation. Very close embankment spacing may cause an unacceptable rise in water levels in the upstream sections and abnormal sand deposition in the upstream or downstream reach. The loss of valley storage can be kept to a minimum if the flood plain on one side can be kept at a lower level or may be left unprotected, depending on the situation. Such a solution is possible only if one of the flood plains on one side have a lower economic value than those on the other side.

The risk of levee breaches cannot be eliminated completely. Fuse plugs should, therefore, be provided in long levees to save the protected areas with high economic values, at the cost of flooding less economically important areas such as farmland.
The area that would be inundated by spillage through breaches should be identified on the basis of previous experience, supplemented by hydraulic studies as necessary. Emergency plans should be devised and warnings should be issued prior to and during major events when such areas are likely to become inundated. Emergency planning for potential breaches of embankments forms a vital component of the integrated flood management approach.

4.3.5 Non-structural measures

4.3.5.1 General

Structural measures alone cannot completely eliminate flood risks. They may even result in generating a false sense of security leading to inappropriate land use in the areas that are directly protected and often in adjacent areas. To reduce flood risk, the vulnerability of economic activities to adverse impacts of flooding also needs to be addressed.

Non-structural measures broadly reduce vulnerability to flooding. They may constitute planning measures and response measures. Flood-plain mapping, land-use planning and regulation, flood forecasting, flood-proofing and insurance are planning and preparedness measures that are to be implemented prior to the onset of floods. Response measures are actions to be taken during and after the flooding; these include fighting floods, emergency evacuation and economic recovery assistance.

4.3.5.2 Land-use planning and catchment management

Land-use planning aims at reducing the risk caused by flooding by addressing the magnitude, exposure and vulnerability of people and their economic activities. Catchment management consists of actions that affect the hydrological process and aim to modify the way or rate in which rainfall is transformed into streamflow, especially floods. Catchment management measures include the introduction of vegetation and crops that protect the soil, the prohibition of cultivation and grazing on steep slopes, reforestation, better forest management and control of shifting cultivation in conjunction with minor engineering works such as check dams, trenches and contour bunds.

Catchment management measures can have a significant impact on small floods and small catchments, but they are much less effective on larger catchments. An important contribution of watershed management is the reduction in silt loading in rivers of aggrading nature. Urbanization caused by land-use change has a significant impact on the magnitude of floods, reducing the time of concentration, and increasing flood peaks, particularly in catchments up to 100 km². Regulating land use through building by-laws can help control urbanization so that it does not seriously affect the hydrological response characteristics of the catchments concerned.

4.3.5.3 Flood-plain regulation

The flood plain is an integral part of the river system which allows the passage of flood flows. When the flood plain is not occupied by water, it forms part of the land system offering possibilities for various economic activities. Integrated flood management should implement patterns of land use which, while taking advantage of the benefits offered by flood plains, reduce to a minimum the damage suffered during the inevitable periods of flooding.

Overdevelopment of the flood plain is the main cause of increasing loss of life and flood damage. Therefore, the most desirable approach is to assess the risks due to flooding, identify them for the information of all stakeholders and, where required, regulate and even prohibit new development in the flood plains by land-use planning and related regulatory measures. However, those developments that are permitted must carry out flood-proof measures for existing and new structures and sometimes attempt to relocate the existing development elsewhere. Where the extent of present development is substantial, or the flood plain is essential for food production or other key economic activities, alternate strategies such as flood-proofing and protection can be considered. Redevelopment and resurrection of an area badly affected by floods can involve permanent alteration of the uses of the land as the only economically feasible alternative, such as resettlement in less hazard-prone areas.

Accordingly, the flood plain may be mapped to show the extent of likely flooding due to floods of different return periods, (for example 1 in 10, 25, 50 and 100 years) by hydraulic routing of design floods of different frequencies through the flood plain and determining the corresponding flood levels, discharges and areas inundated. The results can be drawn onto topographic maps at a scale of 1:20 000 or 1:10 000 or even larger, with contour intervals, depending on the topography.
The unique capabilities of satellites to provide comprehensive coverage of large areas at regular time intervals with quick turn-around times have been valuable in monitoring and mapping past flood events and, therefore, providing information on the flood dynamics for major rivers. Flooded areas, extending to several thousands of square kilometres, can be mapped effectively using satellite data. Multi-temporal satellite data have been used with digital elevation models to identify flood inundation areas, even including flooding under vegetative canopies and, used in conjunction with geographical information systems and terrain modelling, help to identify sections of the inundated flood plains, together with information such as the related water quality.

With the evolution of flood-plain mapping and zoning, appropriate legal and administrative protocols should be developed, including flood-plain regulation and zoning based on by-laws, subdivision regulations, building codes and land development policies (WMO, 2006b).

4.3.5.4 Flood forecasting and warning

Flood forecasting enables society to ascertain the future states of hydrological phenomena, especially as to when the river might inundate its flood plain, to what extent and for how long. Flood forecasts formulated and issued sufficiently in advance allow authorities to respond well in advance by, for example, operating dams; opening or closing gates; making anticipatory releases to enlarge storage capacity; issuing preventive instructions, such as bans on navigation and fishing; invoking emergency measures, such as announcing generalized alerts; mobilizing evacuation of and assistance to the population in high-risk areas or ordering planned breaches of flood dykes. To do so, it is essential to develop and operate flood forecasting and warning systems (see Chapter 7), which would indicate, with sufficient lead time, the expected extent and duration of flooding.

Flood forecasting involves continuous system monitoring and operation, regardless of the frequency of use. If it is to be economical, such a system should, wherever applicable, implement a multi-hazard approach, thereby combining the flood-warning system with other activities, such routine daily weather forecasting, regular hydrometric measurements and navigational traffic control.

Of utmost importance after the formulation of the forecast, is its dissemination to the users or audience concerned as a warning transmitted by telephone, facsimile, radio/wireless/TV bulletins, telegrammes, electronic mail and other media systems (see Chapter 7) for which a robust communication system should be used and well maintained.

4.3.5.5 Flood insurance and other economic instruments

The principal objective of flood insurance is to spread the economic costs of flood damages so that they become more manageable for society. Insurance, unless tied to premium increases on exposure to risk, may not result in reducing the overall losses to society. Flood insurance differs from the other tools for managing flood losses in that, whereas other tools are geared to reducing the cost of flood damage from each flood, insurance distributes the losses over time and space. It places the burden on those who enjoy the benefit of flood-plain location rather than making the burden the sole responsibility of the government.

Risk perception studies carried out in the United States show that without a mandatory component to an insurance system, people tend to perceive flooding as a low risk and therefore do not buy coverage. It is important that policies be consistent, as some people may not purchase insurance if history has shown that the government provides relief to all, regardless of insurance coverage. In addition, a purely voluntary insurance scheme may not yield sufficient funds to cover future compensation claims. However, it has proved difficult to make insurance mandatory – and therefore effective – unless it is preceeded by a major educational campaign. Insurance rates may be tied to risk, with occupants being potentially able to reduce their risk of exposure, for example by flood-proofing their property. Insurance is an option that should be considered but, for the time being, it is probably not a feasible alternative in many developing countries.

Flood insurance is available in a few countries with well-established insurance markets, such as Germany, Japan, the United Kingdom of Great Britain and Northern Ireland, and the United States. There is considerable diversity in the way in which flood insurance is provided, as well as in the methods used to determine premiums. For insurance schemes to be successful, there needs to be a clear definition of the risk, as premiums should reflect the degree of risk at a given location in the flood plain established on the basis of flood frequency and hydraulic modelling. If possible, flood insurance should be considered complementary to a flood-plain zoning programme. There is no single
model of an optimal flood insurance programme for all countries.

4.3.5.6  Flood-proofing

Flood-proofing is defined as follows:

A combination of structural changes and/or adjustments incorporated into the design and/or construction and alteration of individual buildings, structures or properties subject to flooding primarily, for the reduction or elimination of flood damages (USACE, 1995).

An example of a specific action designed to flood proof a structure is the installation of barriers across all openings at ground level to prevent seepage of water and the entry of debris into the main structure. Such devices can be permanent or temporary in design, with the latter being installed preceding the onset of a forecast flood (Szöllösi-Nagy and Zevenbergen, 2005). Flood-proofing can also be achieved by locating structures above the level of the design flood. Such structures could include human dwellings, animal shelters and public buildings, including temporary emergency shelters.

4.3.6  Flood emergency management

No matter what strategies are adopted to reduce flood risk, there will always be some residual risk. Whatever strategies are used to reduce risk from flooding, whether through structural measures and flood embankments or non-structural measures such as reforestation, only partial safety can be promised to those who inhabit the flood plain. When protection fails, damage can be more extensive because of the increased investments made in the flood plain. For many societies throughout the world, the cost of reducing risk by adopting high-cost structural measures or policies aimed at relocating at-risk land use is simply unaffordable. It is also possible that such measures may cause damage to the environment or run counter to the particular development goals. An alternative strategy to be considered, even when structural measures are in place, is to reduce vulnerability through disaster preparedness and flood emergency response. When flooding is inevitable, it is important to take measures that reduce the adverse impacts of such a situation on the lives of people affected. Flood emergency management is aimed at managing and minimizing the damaging effects of flooding. The objective is to prevent loss of human life and avoid the exposure of critical activities by temporarily shifting people and such activities away from flood-prone areas, thereby reducing the negative impacts of flooding on the community. Flood emergency management can be divided into three stages:

(a) Preparedness: pre-flood measures to ensure effective response;
(b) Response: measures taken during the flooding to reduce adverse impacts;
(c) Recovery: measures to help the affected community recover and rebuild after the event.

Emergency management requires cooperation across sectors and administrative levels. In addition to mobilization resources, it is vital to maintain a continuous, timely and precise flow of information in support of those handling the emergency situation. Emergency response planning must be completed well before the flood season and must be based on clear technical and financial plans designed to match scenarios of flood hazards which may occur. These emergency management plans should be the subject of regular review and revision. Lessons learned each flood year need to be incorporated into future plans. Important elements of these plans include the following:

(a) Assessment of flood risk and factors that contribute to losses caused by flooding;
(b) Zoning of protected or unprotected areas according to flood risk;
(c) Inventory of flood control or protection systems;
(d) Analyses of technical means to counteract failure of flood protection structures during floods;
(e) Study of situations which might develop when some elements in the flood protection system fail;
(f) Planning of second, third and subsequent defence lines in the event of progressive failure of linear protection systems such as levees;
(g) Estimate of costs of fighting floods in different situations;
(h) Development of evacuation routes and plans, emergency shelter facilities and provision, medical facilities, and so forth.

Key components of a flood-emergency response plan include an early warning system, protection of critical infrastructures, assessment of immediate needs and provision of safe shelters for the effected population, with adequate facilities for all ages and both men and women.

4.3.6.1  Emergency preparedness and response

The most critical element in flood damage reduction is emergency preparedness and response. As
outlined in the previous section, detailed response plans need to be prepared in advance and reviewed by the coordinating unit with all key agencies and stakeholders, with specific duties being assigned to each so that there will be no confusion under pressure. A coordination mechanism must be included in the plan, including provision for response committees, meeting venues and sources of information. Often this takes the form of an incident management centre where material, support staff and information such as maps and plans are available. The awareness of the affected community should be raised and maintained, with a thorough understanding of how to respond appropriately. This will be critical in achieving quick response in situations such as coordinated evacuation from the affected area when disaster strikes. Information on evacuation routes and emergency shelters should be available to all well in advance. Emergency response teams should receive training early on and their skills upgraded constantly with mock emergency exercises carried out on a regular basis.

A key component of any emergency preparedness plan is an inventory of resources that can be accessed. In the case of flooding, this could include items such as vehicles, buses, trucks, earth-moving equipment, pumps, covering and protecting materials, generators, construction materials and mobile communication equipment. Basic responsibility for developing and implementing such plans generally lies with the administrative authority of the affected area. The same authority must also decide when and how to evacuate the target population, if necessary.

Action taken during floods to prevent damage as well as divert floods from sensitive areas is generally known as flood fighting. This is an emergency measure aimed at mitigating flood impacts on society and the environment. Flood fighting includes building temporary levees with any material that is available, closing breaches with sand bags, moving goods and equipment out of reach of the floodwaters, protecting immovable equipment with plastic sheets or grease, and so forth. When floods occur, water supply and sewerage are often disrupted with potentially serious effects for the health of the population. Therefore, flood fighting includes elements of infrastructure maintenance that are related to public health.

### 4.3.6.2 Post-flood recovery

After the floodwaters have receded, those affected will require assistance to restore pre-existing conditions as soon and as far as possible. Examples of the measures to be taken include the restoration of road and rail links, and the rehabilitation of power installations, public buildings, water supply and sewerage systems, merchandise and shopping areas, industries, factories, poultries, fisheries, piggeries, tube wells and agricultural machinery, irrigation and drainage systems and structures. Action is required to pump water out of low-lying areas and remove overlying sand and silt that will have been deposited on flooded areas. On the whole, efforts are required to provide a post-flood economic stimulus to flood-affected areas.

The relevant administrative agency will provide flood-disaster relief in the form of financial and other aid to relieve the distress of flood victims. At the international level, the United Nations Disaster Relief Coordinator has funds to assist victims of disastrous floods and other natural hazards. In some countries, permanent funds have been established for this purpose and relief may take the form of grants, interest-free or low-interest loans and subsidies. Relief may extend to measures such as the supply of free seeds and other agricultural inputs to farmers. Often aid for flood victims is provided on an ad hoc basis by the government or voluntary organizations such as the local Red Cross or Red Crescent Society at the national level, and by the International Federation of Red Cross and Red Crescent Societies and the Office of the United Nations Disaster Relief Coordinator at the international level. Some governments declare a tax holiday for those affected, thereby further reducing the burden on them.

After a major flood, it is very important and urgent to make an assessment of the causes and effects of the disaster and of the performance of emergency actions, followed by recommendations that would improve preparedness and reduce flood losses for the next event. One thing is certain: there will always be another flood event at some time in the future.

### 4.4 Irrigation and Drainage

#### [HOMS K70]

#### 4.4.1 Irrigation

The practice of irrigation as a means of producing food has been around for over 5 000 years (Framji, 1987). During the second half of the twentieth century, the total irrigated area in the world increased from about 115 million hectares to over
270 million hectares. This has led to a more than twofold increase in the world's total food grain production (cereals, oilseeds and pulses), from 1,763 million tonnes to 3,891 million tonnes. Irrigated land accounts for 20 per cent of the total crop area in the world and over 40 per cent of total food production. Irrigation, combined with the use of high-yielding crop varieties, which can be grown only under irrigated conditions, has indeed been a crucial element in many countries' struggles to achieve and maintain self-sufficiency in food grain production.

Irrigation is the largest user of water, taking more than 70 per cent of the world’s fresh water supply. Although irrigation is not a new practice, most irrigation systems are operated inefficiently, with efficiency seldom exceeding 40 per cent. History abounds with examples of civilizations that owe their success to well-planned and well-managed irrigation systems and those which met their downfall because of improper and inefficient management of irrigation systems. Efficient management of irrigation systems centres around maintaining an appropriate soil moisture regime in the plants' root zone to promote healthy plant growth. This requires the timely supply of adequate amounts of water and removal of excess water from the root zone. Therefore, both irrigation and drainage are necessary for the proper management of water for agriculture.

4.4.1.1 Why plants need irrigation

Water is essential for plants in many ways:
(a) Nearly 70 per cent of a plant is comprised of water;
(b) Initially, water is required to soften the seed and its covering to facilitate emergence first of the root and then of the seedling above the soil;
(c) Water functions as a solvent and dissolves and transmits through the plant roots nutrients such as nitrogen, phosphate and potassium, which are essential for healthy plant growth;
(d) Water is the solvent for biochemical reactions in plants, such as carbon fixation and photosynthesis;
(e) The carbon, nitrogen, hydrogen and oxygen required for plant growth are derived from water and atmospheric air and make up most of the body of the plant;
(f) Roughly 95 per cent of the water absorbed by the plants is transpired from the leaves and the stems. This process also helps to cool the plants during hot weather;
(g) Without water, plants wilt and ultimately die.

The soil is capable of retaining moisture with the forces of adsorption and surface tension. Any additional moisture that enters the soil medium, beyond that held by these two forces, moves down through the soil pores under the influence of gravitational force. This process is known as percolation. A measure of the tenacity with which water is retained in the soil, indicating the force required to extract water from the soil, is referred to as the soil moisture tension. The amount of moisture in the soil medium is referred to as the soil moisture content. The soil moisture content at which plants can no longer extract water from the soil to meet their evapotranspiration requirements is known as the wilting point. When the soil moisture falls to this level, plants wilt and die unless water is replenished in the root zone. The amount of soil water available between the moisture content at the field capacity and at the wilting point is referred to as the available moisture capacity.

The purpose of crop irrigation is to ensure that an adequate water supply in the root zone at all times, in the range between field capacity and wilting point. Soil moisture is affected by rainfall, irrigation, evapotranspiration, runoff, infiltration and deep percolation. When all the interstices in the soil are completely filled with water, the soil is said to be at its saturation capacity. In this state, water will drain out of the soil root zone under the influence of gravity until an equilibrium is reached. The soil is then said to be at field capacity. This stage is generally reached within one to three days after irrigation or rainfall. Efficient irrigation returns root zone to field capacity. Water applied in excess of this amount is considered wastage unless deliberately done for leaching purposes.

4.4.1.2 Crop water requirements

Crop water requirement is defined as the depth of water needed to meet the water loss through evapotranspiration of a disease-free crop growing in large fields under conditions which impose no soil, soil water and fertility conditions, thus achieving full production potential under the given growing environment (Doorenboes and Pruitt, 1977).

The water requirement is crop- and location-specific, and is influenced by crop species, local climate and the soil. It is estimated for a specified period of time, for example a week, month or growing season.

The evapotranspiration requirement comprises evaporation from the adjacent soil surface, evaporation from the intercepted water and transpiration from the stomata of the epidermis of the plant.
surface such as bark or leaves. In addition, water is also required for the metabolic activities for plant growth. The total water required for healthy crop growth is referred to as the consumptive use. However, the water needed for the metabolic activity is very small – less than one per cent – compared with the evapotranspiration requirement and, as such, the terms consumptive use and crop evapotranspiration are used interchangeably.

Part of a crop’s water requirement is often met by the local rainfall and a contribution from the soil moisture storage, as well as through the capillary rise of groundwater wherever the groundwater table is nearer to the root zone. Only a portion of the local rainfall, called effective rainfall, is used by the crop for its growth. Care must be taken with the use of this term because effective rainfall means different things for practitioners of different disciplines. For a water resources engineer, effective rainfall is the rainfall that reaches the storage reservoir as runoff, while for geohydrologists it is the portion of the rainfall that contributes to groundwater storage. For an agronomist or farmer, however, it is the portion of the rainfall that contributes to crop growth. An agronomist or farmer, however, it is the portion of the rainfall that contributes to meet the crop’s evapotranspiration requirement. In terms of crop water requirements, effective rainfall is defined as that part of the rainfall which is useful directly or indirectly for crop production at the site where it falls, but without the use of mechanical means. The remaining rainfall either evaporates back into the atmosphere, runs off the soil surface, or is absorbed by the soil or percolates through the root zone. The amount of effective rainfall depends on various factors such as plant species, soil moisture conditions in the root zone, climate and the time distribution of rainfall. Details of the estimation of effective rainfall for crop-soil-climatic-specific situations are discussed in Irrigation and Drainage Paper 25 of the Food and Agriculture Organization of the United Nations (FAO)(Dastane, 1972).

4.4.1.3 Determination of crop water requirements

Over many years, FAO has issued various guidelines on the estimation of crop water requirements. In particular, Irrigation and Drainage Paper 56 (Allen and others, 1998) contains a detailed computation of crop water requirements.

In 1990, a panel of experts recommended to FAO the adoption of the Penman–Monteith combination method as the new standard for reference evapotranspiration estimation. This method uses standard climate data that can be easily measured or derived from other commonly measured data and is reported to provide consistent values for crop water requirement calculations through the world. Basic definitions and concepts involved in the determination of crop water requirements are given briefly below.

4.4.1.3.1 Evaporation and transpiration

Evaporation is the process of converting liquid water into water vapour and its removal from the evaporating surface. Evaporation occurs from lakes, rivers, wet surfaces, soils and vegetation. Transpiration is the process of vaporizing liquid water contained in plant tissues and removing it to the atmosphere. Crops predominately transpire through stomata. Nearly all the water taken up by plants is lost by transpiration and only a tiny fraction is used within the plant for its metabolic growth.

4.4.1.3.2 Evapotranspiration

Evaporation and transpiration occur simultaneously from a cropped area and it is very difficult to distinguish between the two. Hence the two are represented together by the term evapotranspiration (ET). As a rule, the units of evapotranspiration are expressed as mm per day. Evaporation from a cropped soil depends mainly on the amount of solar radiation reaching the soil surface and varies with the stage of crop growth. At sowing stage, nearly 100 per cent of evapotranspiration comprises evaporation only, while with full crop cover, more than 90 per cent of evapotranspiration comes from transpiration. The crop type, variety and growth stage should be considered when assessing the evapotranspiration from crops. Variations in crop height, crop roughness, crop rooting characteristics, albedo, resistance to transpiration and ground cover lead to different evapotranspiration values for crops under identical environmental conditions.

Three terms are used to express evapotranspiration: the reference evapotranspiration (ET₀), crop evapotranspiration under standard conditions (ETc) and crop evapotranspiration under non-standard conditions (ETc). For more information on evaporation and evapotranspiration, see Volume I, Chapter 4, of the present Guide.

4.4.1.3.3 Reference evapotranspiration

The evapotranspiration rate from a reference surface that is not short of water is called the reference evapotranspiration, ET₀. The concept of the
reference evapotranspiration facilitates the study of the evaporative demand of the atmosphere independently of soil factors, crop type, crop development and management practices. Thus the only factors affecting reference evapotranspiration are climate parameters; hence it can be computed from observed or estimated weather data. The FAO Penman–Monteith method is recommended as the sole method for determining reference evapotranspiration. The method has been selected because it closely approximates grass reference evapotranspiration at the location evaluated, is physically based, and explicitly incorporates both physiological and aerodynamic parameters. The method requires radiation, air temperature, air humidity and wind speed data. Calculation procedures to derive climatic parameters from meteorological data are presented. Procedures to estimate missing meteorological variables required for calculating reference evapotranspiration are also outlined. This allows for the estimation of reference evapotranspiration under all circumstances, even where climate data is missing. Relating evapotranspiration to a specific surface provides a reference to which evapotranspiration from other surfaces can be related. This obviates the need to define a separate evapotranspiration for each crop and stage of growth. Such a reference value also facilitates a comparison of values of reference evapotranspiration at different locations or in different seasons. The reference surface is a hypothetical reference crop with specific characteristics. The reference crop is defined as:

..a hypothetical crop with an assumed height of 0.12 m, with a surface resistance of 70 s m$^{-1}$ and an albedo of 0.23, closely resembling the evaporation from an extensive surface of green grass of uniform height, actively growing and adequately watered... (Allen and others, 1998).

Detailed calculations of the reference crop evapotranspiration are given in Chapter 4, Part A of FAO Irrigation and Drainage Paper 56 (Allen and others, 1998). Use of other denominations such as potential evapotranspiration (PET) is discouraged due to certain ambiguities associated with such terms.

4.4.1.3.4 Crop evapotranspiration under standard conditions

This refers to the evaporation demand of crops that are grown in large, adequately irrigated fields under excellent management and environmental conditions, and achieve full production under the given climatic conditions (Allen and others, 1998).

4.4.1.3.5 Crop evapotranspiration under non-standard conditions

Actual crop evapotranspiration is affected by factors such as soil salinity, presence of hard pans in the subsoil, poor soil fertility and soil management, inadequate plant protection measures, ground cover, plant density and soil water content. Hence under such non-standard conditions, crop evapotranspiration under standard conditions generally requires an adjustment. The prediction of the reduction in evapotranspiration caused by soil water salinity may be achieved by combining yield–salinity equations from FAO Irrigation and Drainage Paper 29 (Ayres and Westcot, 1985) with yield–evapotranspiration equations from Irrigation and Drainage Paper 33 (Doorenboes and Kassam, 1979). These details are given in FAO Irrigation and Drainage Paper 56 (Allen and others, 1998).

The crop evapotranspiration, $\text{ET}_c$ for any particular crop is determined by multiplying the reference evapotranspiration $\text{ET}_0$, with a coefficient $K_c$, called the crop coefficient ($\text{ET}_c/\text{ET}_0$). The value of the crop coefficient is crop specific and is dependent on the stage of growth of the crop and the prevailing weather conditions. The differences in the crop canopy and aerodynamic resistance relative to the reference crop are also accounted for in the coefficient. As such, the coefficient serves as an aggregation of the physical and physiological differences between crops.

4.4.1.4 Irrigation requirement

Irrigation is defined as the artificial supply of water to plants to ensure the healthy growth of a crop. It stands in contrast to the natural supply from rainfall, soil moisture and capillary contribution from groundwater, among others. The net irrigation requirement is the amount of irrigation to be provided to the plant root zone after accounting for the contribution of rainfall, soil moisture and capillary supply from groundwater. It should include any special requirements such as those for the leaching of salts from the root zone and, in the case of rice paddies and jute, the water required for land preparation, standing water requirements, percolation and periodical draining.

Accordingly, the net irrigation requirements of a crop is equal to the evapotranspiration requirement $\text{ET}_c$ at its root zone (the crop water requirement), plus special crop requirements such as water required for leaching and land preparation minus effective rainfall, plus the contribution of
soil moisture and the capillary supply from groundwater.

Supplying irrigation to a crop inevitably incurs some water loss between the source and the root zone as a result of the transport, distribution and application of the water. The efficiency of irrigation depends on the efficiency of the conveyance system, the distribution system and the particular method and timing of irrigation application. Accordingly, the total amount of irrigation water required, known as the gross irrigation requirement, is assessed as follows:

\[
\text{Gross irrigation requirement} = \frac{\text{Net irrigation requirement}}{\text{Irrigation efficiency}}
\]

where irrigation efficiency is the application efficiency times the distribution efficiency times the conveyance efficiency.

The net and gross irrigation requirements can be assessed at the individual field or farm level as well as at the level of command area of an outlet or minor or major distributary branch or main canal, or an irrigation project using the corresponding value of efficiency. The gross irrigation requirement is expressed in terms of volume of water per hectare of cropped area over a specified period of time such as a week, month or crop season.

### 4.4.1.5 Irrigation systems

Sources of irrigation water can be stored in man-made storage reservoirs or lakes, groundwater developed locally through open wells or tube wells, or through diversion of natural channels. In the case of storage reservoirs formed behind dams and diversion weirs, irrigated areas may be far from the source and water may need to be transmitted through a large distribution network of canals and major and minor distributaries before it is delivered to the farmers' fields (see Figure II.4.11). The distribution network is generally minimal for water sourced from river diversions, lakes and groundwater wells, in descending order. Considerable seepage losses are associated with conveyance of water from the source to the field outlets. Additional water losses are associated with distribution of water below the outlets through the water courses and field channels. It is therefore important that the irrigation efficiency used for assessing the gross irrigation requirement at the source take account of conveyance and distribution losses in the system.

Irrigation is applied to crops by various methods that can be broadly classified as surface and subsurface methods. The irrigation methods can be further classified as irrigation based on gravity flow or based on pressurized water flow (see Figure II.4.12). Detailed descriptions and design procedures for

![Figure II.4.11. Irrigation system](image-url)
these irrigation systems are available in standard textbooks on irrigation.

4.4.1.6 Soil moisture

Different methods of irrigation water application give rise to different patterns of moisture availability in the crop root zone (see Figure II.4.13). The irrigation method that produces uniform root zone moisture content at or near the field capacity throughout the crop season causes the least stress to the plant, thereby facilitating healthy growth. Modern irrigation control and application techniques are based on monitoring the soil moisture status in the root zone. The soil moisture content at different depths in the root zone can be determined through gravimetric or volumetric methods. Modern tools such as the neutron moisture probe and the time domain reflectometer are being increasingly used for accurate monitoring of the soil moisture content in the root zone (see Volume I, 4.5). Information from soil moisture probes can be directly entered into computers which assess the irrigation requirement and operate the irrigation system automatically. Most of the modern automated irrigation control and operating systems are based on this approach.

4.4.1.7 Irrigation scheduling of crops

Irrigation water is to be applied in such a way that, as far as possible, the crop water requirements are met over time. As the crop water requirements change in time with the stage of growth of the crop, as well as with the occurrence of rainfall, the supply of irrigation water to a crop should follow a well-planned schedule. Such a schedule should ensure the application of the right amount of water to the crop at the right time so as to obtain high yield of good quality produce, with high water use efficiency and with least damage to the environment, all at a low cost of operation. Determining such a schedule is referred to as irrigation scheduling. There are several practices in vogue for irrigation scheduling. Irrigation scheduling procedures vary depending on whether there is an adequate supply of water or the supply is limited.

4.4.1.7.1 Irrigation scheduling under adequate water supply

When an adequate supply of water is available, the objective of irrigation scheduling is to eliminate periods of water deficit so as to achieve the full potential crop yield. Irrigation doses are applied to replenish the soil moisture whenever the soil water content of the root zone falls to a level at which it begins to have an adverse impact on crop yield. The agronomists or crop scientists seek to maximize the output of the individual crop fields for a given water supply using the empirically derived scientific knowledge of crop response to available soil water. The main factors which govern the irrigation schedule in this case are climate, soil, type of crop and its stage of growth. Numerous agronomic studies are reported in the literature describing irrigation

Figure II.4.12. Broad classification of irrigation methods

Figure II.4.13. Soil moisture regimes in different irrigation methods
schedules for a large variety of cropping systems and crops: The studies are generally focused on determining the depth and intervals of irrigation based on crop water requirements during different stages of the crop growth, soil moisture extraction patterns from the root zone, optimal soil moisture regimes to be maintained in the root zone at various growth stages and other factors.

These procedures are generally guided by one of the following criteria:
(a) Critical growth stages of the crop (Prihar and others, 1976);
(b) Ratio of irrigation water applied to the cumulative value of pan evaporation (Prihar and others, 1976);
(c) Soil-water depletion (Rao and others, 1988a and 1988b; Hajilal and others, 1998);
(d) Assessment of crop evapotranspiration using climate factors and crop factors derived from experiments conducted locally for each crop and based on the Penman–Montieth method (Doorenbos and Pruitt, 1977; Allen and others, 1998);
(e) Soil-moisture tension values observed in the field;
(f) Visual crop features: plant drooping, change in leaf colour, rolling of leaves and so forth;
(g) Plant indices such as relative leaf water potential, leaf water content and leaf water diffusion resistance.

4.4.1.7.2 Irrigation scheduling under limited water supply

When water supplies are limited, crop water deficits in some periods of the growing season are unavoidable. Crop response to deficits at different periods of the growing season is not uniform and deficits in some critical periods of growth have a greater adverse impact on yield than in others. Therefore, under limited water supply conditions, the irrigation scheduling problem becomes one of distributing the deficits over the crop growing season in such a way that they have minimum impacts on crop yields. The problem is complex and any attempts at its solution require integration of information on soil, growth stage of the crop and crop responses to timed inputs of water. Such a framework consists essentially of the development and incorporation of the following three components:
(a) A soil-water balance model to break down the water inputs (rainfall and irrigation) into different components;
(b) A dated water production function model of crops to relate crop yield to water used at different periods of crop growth;
(c) An optimal irrigation programming model that incorporates (a) and (b).

4.4.1.7.3 Optimal irrigation schedules

Irrigation programming refers to the process of drawing up an optimal schedule of irrigation applications for a crop during its growth period under specified conditions of water availability and climate. This calls for exploration and evaluation of the effect of all possible irrigation regimes on the crop yield and establishing an optimal regime using an appropriate mathematical optimization programme. The dated water production functions that relate the yield response to timed irrigation applications over the crop season provide a means for this and incorporate effects of both timing and quantity of irrigation water application on crop yield. The time periods are either chosen to coincide with the physiological growth stages of crops or are taken as some convenient time period such as a month or a week.

The main difficulty addressed by these models is that irrigation decisions in different intervals of the growing season are not independent. Each irrigation decision is based on available soil moisture, crop status and available water supplies for the remaining period of the growing season. Information about all of these factors up to the time of the decision must be utilized before the decision to irrigate is made. Those decisions are thus multistage, sequential and state dependent. Basically, these models simulate the various dynamic processes that lead to crop production and yield and their changes in response to the changes in environmental conditions of which water stress is one. Near-potential yields can be obtained by proper choice of irrigation schedules even under relatively high water supply deficits (Rao and Rees, 1992).

Despite advances in irrigation scheduling models, human judgment and expertise will continue to be a major source of decision support in irrigation management. The most appropriate irrigation schedule can be developed by using quantitative model predictions together with local knowledge and experience of farmers and irrigation operators. Such a balance is possible by integrating models and heuristic knowledge in an expert system framework.

Many farmers grow a number of crops in the same season. In such situations, a limited water supply implies that water is not adequate to produce potential yields of all the crops. This leads to competition
for water between crops, both at the seasonal and intraseasonal level. The problem of multicrop seasonal and intraseasonal allocation of water can be solved by dividing the problem into two levels, seasonal and intraseasonal.

4.4.1.7.4 Irrigation scheduling in real time

Irrigation schedules can be based on optimization models for planning and design purposes. However, in real-time operation, both weather and water supplies may be different from those assumed in deriving the planned crop irrigation schedules. Hence, the optimized schedules may need to be modified to match the real-time information on weather and water supplies. Since the effect of each decision can be evaluated as crop yield only at the end of the season, irrigation decisions should be developed in a sequential manner while going forward in time. Therefore, an irrigation decision is to be made each week with the entire planning horizon in mind.

4.4.1.7.5 Use of medium-range weather forecasts in irrigation scheduling

Medium-range weather forecasts provide information about the weather 3 to 10 days in advance and can be used in agricultural management including irrigation scheduling. Historical rainfall data can be used to examine the influence that the three-to-five-day advance information on rainfall will have on the irrigation scheduling of crops. While medium-range weather forecasts are useful for irrigation scheduling regarding shallow soils and situations in which small irrigation depths are applied frequently, they do not lead to significant water savings for deep-rooted crops in soils with relatively highly available water capacity.

4.4.1.8 Irrigation methods

4.4.1.8.1 Traditional irrigation methods

The operational efficiency of traditional flow irrigation systems is low; therefore, there is a great need for adopting modern, efficient irrigation methods. Alternate furrow irrigation, surge flow irrigation and pressurized irrigation systems (drip and sprinkler irrigation) are considered to be efficient technologies.

4.4.1.8.2 Surge irrigation

Surge flow irrigation is a recent surface irrigation method (Stringham and Keller, 1979; Stringham, 1988). This is accomplished by surging the water down the furrows at timed intervals until the water reaches the end of the furrow. In the past few years, field researchers have investigated significant new roles for surge flow that rely on its ability to distribute water uniformly, save water, reduce infiltration and deep percolation losses, and control runoff and drainage through surface systems. This method, which applies water uniformly, is used to create a shallow, uniform water profile and keep the water at the root zone cutting down deep percolation. It has an efficiency of 85 per cent, and saves up to 25 per cent in fertilizer costs.

4.4.1.8.3 Pressurized irrigation systems

As the water is conveyed through a pipe system, the conveyance losses are eliminated, resulting in higher irrigation efficiencies. Pressurized systems are recognized as achieving high water-use efficiency and improved crop productivity with low labour inputs and adaptability to hilly terrain. They are suitable for water-scarce areas, can reduce frost attack and can readily apply water-soluble fertilizers. The system is well suited to canal, tank and groundwater irrigated areas. All close-grown crops such as cereals, pulses, oil seeds, sugar cane, cotton and other plantation crops can be grown using the sprinkler irrigation method. An advantage of these systems is that undulating lands and shallow soil areas can be irrigated without having to level the land.

4.4.1.8.4 Drip system

The drip system of irrigation is a comparatively modern method of water application. The initial investment is costly, but is the drip system is suitable for situations calling for high water-use efficiency and involving undulating terrain. Considerable experimental research has been carried out over the past 30 years to investigate water savings and yield increases, design of appropriate components and their materials, moisture distribution and irrigation, and fertilization under drip irrigation. Water application efficiencies of 80 to 90 per cent can be achieved with this method.

4.4.1.8.5 Sprinkler system

Sprinkler systems distribute water in a manner similar to rainfall, so that the runoff and deep percolation losses are minimized and uniformity of application is close to that obtained under rainfall conditions.

4.4.1.8.6 Microsprinkler system

Microsprinklers facilitate spraying of water under the tree canopy around the root zone of the trees,
about 30 cm high, and work under low pressure. This method is least affected by wind. The exact quantity of water required can be delivered daily to each plant at the root zone. Water is given only to the root zone area as in drip irrigation but unlike the much wider distribution provided by sprinkler irrigation. This method is well suited to the watering of trees, orchards and vegetable crops, particularly in combination with the use of local renewable energy sources for pumping water.

4.4.1.8.7 Low-energy precision application systems

Recent innovations in microsprinkler systems are low-energy precision application systems. In these, the laterals are equipped with drop tubes fitted with very low pressure orifice emission devices called socks. Water is discharged just above the ground surface into dead-end furrows or microbasins, thus preventing soil erosion and runoff. These systems are not affected by wind forces and, in addition to saving considerable energy, they provide uniformity of application and very high application efficiencies, in the order of 98 per cent.

4.4.1.9 Development of decision-support systems and use of geographical information systems in irrigation

It is useful to link simulation models and system models to spatial databases by means of a geographical information system so as to develop expert decision-support systems for conjunctive use and real-time irrigation operation. This approach focuses on providing decision support to irrigation planners and managers, enabling them to use routinely collected spatial data and forecasts more effectively.

4.4.1.9.1 Geographical information systems for spatial distribution of recharge

The spatial distribution of recharge for variable weather, soil, land-use and water-supply conditions over the command area of an irrigation project can be assessed using a geographical information system. A new coverage can be derived by superposing digital maps of the command area with different map coverage, such as those for rainfall, groundwater and cropping patterns. Each of the polygonal areas of this coverage will be homogeneous with respect to all the coverage used. As such, these polygons can be used as the basic units for water balance studies and irrigation scheduling (Chowdary and others, 2003).

4.4.1.9.2 Development of decision-support systems for real-time irrigation management

Decision-support systems can be developed for the real-time management of irrigation systems by suitably combining the real-time data with the decision-support-system scheme developed to plan irrigation system management. A simple soil-water balance model can be used to assess the root zone soil moisture condition and a simple canal flow model can be used to account for seepage losses. Based on this information and on knowledge of the water available in the distributary and the medium-term weather forecasts, it is possible to derive the bi-weekly irrigation requirements at the head of each distributary. This information can then be linked to the geographical information system of the command area canal system to facilitate the following tasks:
(a) Selecting the distributary of interest from the canal network;
(b) Running the field water balance model in real-time;
(c) Drafting a report of the current water status;
(d) Preparing a water indent for the irrigation requirements at the head of the distributary.

4.4.1.10 Conjunctive use of surface and groundwater in irrigation

Conjunctive use refers to the integrated management of surface and groundwater resources in a harmonious manner so that the best use of both water sources is achieved to meet specified objectives in the area. For improved water-use efficiency in canal-irrigated command areas, optimal and efficient utilization of surface water and groundwater becomes imperative and should be ensured from the planning stage. For example, using surface water during the monsoon period and groundwater during the non-monsoon period to irrigate the same land mass is a type of conjunctive use. Similarly, seepage from the canals and percolation of irrigation water both contribute to groundwater storage which can be withdrawn at a different point in time for irrigation. This is another, albeit inadvertent, example of conjunctive use. Conjunctive use can help to achieve the following aims:
(a) Increase the availability of water supply for irrigation;
(b) Enhance sustainability of the long-term groundwater regime equilibrium;
(c) Improve regulation and facilitate the phased development of a water resource, using the storage space of the aquifer;
(d) Provide flexibility in supply to match the water demand by smoothing peaks in surface water supplies;
(e) Reduce waterlogging and soil salinity.
4.4.1.10.1 Guidelines for conjunctive use

Irrigation planning for conjunctive use requires consideration of quantitative and qualitative aspects of groundwater and surface water resources as well as economic aspects. Putting conjunctive use into operational practice requires the development of guidelines (CWC, India, 1997) which may include the following tasks:
(a) Mapping of groundwater conditions and their changes in time and space;
(b) Quantification of available groundwater resources in the region based on detailed water balance studies;
(c) Assessment of the additional recharge to groundwater;
(d) Estimation of minimum desirable and maximum permissible limits to additional extraction of groundwater for conjunctive use purposes;
(e) A broad water-use plan, based on existing water availability conditions;
(f) Planning the regulated combined use of groundwater and surface water in time and space;
(g) Identifying and detailing the areas to be served from the surface water and groundwater sources separately or in combination;
(h) Assessing the adverse socioeconomic impacts of conjunctive use in the long term.

4.4.1.11 Use of marginal quality water for irrigation

Water is considered suitable for irrigation when it has no negative osmotic or specific toxic effects on crop production, contains no solute affecting the chemical or hydraulic properties of soil and does not cause deterioration of groundwater or surface water. These adverse conditions are caused primarily by salt accumulation in the root zone of plants. Accordingly, water of marginal quality may be used during stages of growth that are less sensitive to poor-quality water, especially salinity, and by ensuring that there is no accumulation – or as little as possible – of salts in the root zones. This can be prevented, either by leaching with a regular supply of adequate water or by adopting special irrigation methods. In situations of inadequate water availability or of water salinity, the drip and pitcher irrigation methods are the most appropriate. These methods ensure that the salts do not accumulate near the roots and maintain low soil-moisture tension, thus protecting the plants from adverse effects.

Different qualities of water can also be used in arid climates by blending the marginal quality water with good-quality water in the supply system to produce a predetermined quality to match the salt tolerance of the crop, or through alternate irrigation with good- and marginal-quality waters from different sources, such as canal water and saline groundwater.

Crops have different salinity tolerance levels. When salinity cannot be maintained at acceptable levels by using the above methods, it is desirable to choose crops or varieties that are tolerant to salinity, such as vegetables, barley, sorghum, wheat and tomato, and to adopt suitable soil and water management practices, along with a judicious use of fertilizers.

4.4.2 Agricultural drainage

Agricultural drainage is the removal of dissolved salts and excess water from the root zone and land surface to create more favourable plant growth conditions. Agricultural land drainage by surface and subsurface drainage systems was reportedly practised by Egyptians and Greeks in prehistoric times. For most irrigation projects throughout the world, drainage needs have not been adequately assessed and handled. Failure to realize the potential benefits of irrigation projects is often attributable to inadequate attention to drainage. The cost of drainage is often significant and acts as a deterrent to investment in the initial planning stages and implementation of irrigation projects. The adverse effects of inadequate drainage begin to appear only after several years of operation of an irrigation system. Attention to drainage at this stage is generally too late and hence ineffective.

4.4.2.1 Purpose of agricultural drainage

Waterlogging of agricultural land is, in a broad sense, the condition of saturation of the crop root zone leading to restricted aeration, reduced oxygen levels and increased carbon dioxide levels. Under hot, arid conditions, the evaporation process brings up shallow subsurface water along with dissolved salts to the soil surface and can render the soil saline after many years. Conditions of waterlogging and soil salinity are detrimental to healthy crop growth and lead to reduced agricultural production. The reasons for waterlogging and soil salinity are many, and include high rainfall, unfavourable topography, lack of natural drainage, low-permeability soils, soils with hard pan at shallow depths, seawater intrusion, high evaporation during long, hot and dry periods, and the presence of salts in the soil. In addition, many anthropogenic activities
accentuate the problem, such as the inappropriate management of land and irrigation water, use of poor-quality water for irrigation, high seepage from irrigation systems, adoption of unsuitable cropping patterns and blockage of natural drains and outlets due to the construction of roads, culverts, bridges and railways. Most soils in arid regions contain some salts. India, Indonesia, Iraq, Egypt and Pakistan, for example, have vast tracts of waterlogged, saline lands. Reclamation of such lands is costly and has low economic returns.

The objective of agricultural drainage is to improve the physical and chemical environment of the land so as to enhance its productivity or maintain it at a high level. This is achieved by removing excess surface and subsurface water, together with dissolved salts. The water to be removed may be excess water applied through irrigation, excess rainfall and seepage from conveyance or storage systems or irrigated areas upstream. Most agricultural lands have some degree of natural surface and subsurface drainage. Artificial drainage is achieved by installing surface and subsurface drainage systems to achieve the following:

(a) Maintain a correct water and nutrient balance in the agricultural lands;
(b) Remove excess water and stimulate healthy crop growth;
(c) Restore root zone aeration;
(d) Remove excess salts through surface disposal or leaching;
(e) Increase the availability of applied nitrogen fertilizer by minimizing denitrification;
(f) Reduce the specific heat of the soil-water medium;
(g) Lower the water table;
(h) Increase the root zone from which nutrients can be absorbed.

### 4.4.2.2 Types of drainage

Drainage systems can be classified as surface, subsurface or vertical.

#### 4.4.2.2.1 Surface drainage systems

Surface drainage is the removal of excess water from the land surface through gravitational flow involving mainly open drains and land grading to prevent surface water stagnation. The disposal of excess water is achieved by installing a network of surface drains that link the area to be drained with the main outlet. A hierarchical pattern is usual in which the smallest component of the system is the field drain, followed by the lateral drain, the collector drain, the sub-main and the main drain (see Figure II.4.14). In some situations, isolated waterlogged patches of land may be drained through randomly located drains. The field drains are small, temporary and shallow (<15 cm deep), with a gentle slope towards the lateral drain.

Additional drains, such as seepage and interceptor drains, may also be deployed independently of the main drainage system. Seepage drains are aligned with and adjacent to the source of seepage, such as a canal or a drainage channel. Interceptor drains serve to intercept surface or subsurface flow from higher reaches before it submerges the cultivated lands at lower elevations. Surface drainage is more suitable for shallow soils and those with low permeability. While surface drainage systems are less expensive, they require periodic maintenance.

#### 4.4.2.2.2 Subsurface drainage systems

Subsurface drainage involves the removal of excess water held at or near the crop root zone so as to control the level of the groundwater table and reduce soil salinity. Subsurface drainage systems consist of moles or buried perforated pipes or tiles laid sufficiently below the crop root zone. The free water in the saturated soil profile, along with the dissolved salts, flow into the subsurface drains which, in turn, discharge into an outlet or a collector drain. Wherever the topography is not conducive to the disposal of drainage water by gravity, a sump well and a pump are provided at the end of the collector drain to permit removal of the drainage water. The hierarchical pattern of the surface drainage system is equally valid for subsurface drainage systems (see Figure II.4.15).

Subsurface drains are more suitable for soils with high permeability. Fine-textured soils have low permeability; owing to their small pores, they become easily
clogged with colloidal material, obstructing the gravitational flow into the drains and rendering the drains ineffective. Subsurface drainage systems in irrigated lands in arid and semi-arid regions are suitable for leaching dissolved salts from the root zone. As subsurface drains are laid well below the land surface, there is no loss of cultivable land area. However, while the initial cost of a subsurface drainage system is greater than that of a surface drainage system, the maintenance costs are practically negligible and the operational life is much longer.

4.4.2.2.3 Vertical drainage systems

A vertical drainage system involves the mechanical pumping of the water through a shallow tube-well suitably designed and installed in the field. A multiple well-point system, comprising a network of closely spaced shallow tube wells, can also be used to provide drainage of a waterlogged region, particularly where salt water from deeper layers is likely to be pumped up if a single tube well of higher discharge capacity is used. The result of operating such a system is that all the individual cones of depression will interlink and draw down the water table under a larger area. Generally, all the wells are joined to a single pump. The pumping rate is decided according to the safe depth at which the groundwater table is to be maintained. The system can drain excess water from depths of two metres, which is the normal limiting depth of subsurface drain systems. Vertical drainage calls for the expensive construction of tube wells and a continuous energy supply for pumping.

Another form of vertical drainage involves induce-ment of evapotranspiration by planting appropriate vegetation over the area to be drained. Plantations such as eucalyptus, poplar and casuarinas, which transpire at a high rate, are being used for this purpose. This is also referred to as biodrainage. Biodrainage is found to be especially appropriate for landlocked areas where suitable outlets for disposal of drainage water do not exist or are limited in capacity. This recent technology requires further testing and evaluation to determine its suitability in specific situations.

4.4.2.3 Design of agricultural drainage systems

This involves the following steps:

(a) Surface drainage
   (i) Determining the quantity of excess water to be drained;
   (ii) Deciding on the rate at which the excess water is to be drained;
   (iii) Designing the physical components of the drainage system: selection of suitable outlet location based on the knowledge of existing outlets and disposal systems (natural streams), layout and sizes of the drains, design of outlet and ancillary control structures;

(b) Subsurface drainage
   (i) Determining the quantity of excess water to be drained by finding the amount of recharge by rainfall or excess irrigation;
   (ii) Determining the hydraulic head under steady- and unsteady-state water table conditions;
   (iii) Designing physical components of the drainage system. This includes the determination of the layout and the sizes of the drain pipes, the depth at which the pipes are to be located, slopes and alignment of pipes, location and selection of the outlet and so forth.

The design of a drainage system is based on the amount of water to be removed from an agricultural area in one day so as to avoid damage to the crops due to waterlogging. It is referred to as the drainage coefficient and is expressed in terms of centimetres per day or in litres per second per hectare. The value of the drainage coefficient is a function of the rainfall characteristics, such as intensity and duration, the rate of runoff generated, the crop tolerance to excess water and the stage of crop growth.

The drainage coefficient is the key parameter in the design of surface and subsurface drainage systems. In the case of subsurface systems under steady-state conditions, the coefficient has the same meaning as for surface drainage systems. However, for unsteady
flow conditions, the concept of the drainage coefficient for subsurface systems is different in the sense that it is the rate at which the water table is to be lowered.

Accordingly, soil-water properties such as infiltration, saturated hydraulic conductivity and drainable porosity play an important role in the design.

The depth at which subsurface drains are placed is decided on the basis of the maximum depth of the root zone and the capillary rise of water in the soil which, in turn, depend on the soil texture.

Details of design practice and operation of surface and subsurface drainage systems can be obtained from standard text books on drainage. The International Commission on Irrigation Drainage has produced several publications on the subject.

4.4.3 Use of remote-sensing and general information systems in irrigation and drainage

Recent developments in remote-sensing technology are proving valuable in the planning and monitoring of irrigation and drainage systems. Remote-sensing can be used to identify land use and areas that are cropped, irrigated, waterlogged or flooded. It can also yield information on soil salinity, crop water needs and stress, and crop yields. Information derived from remote-sensing techniques, linked to a geographical information system, is considered to be the future for planning and managing irrigation and drainage systems.

Landsat Thematic Mapper (TM) and SPOT Multispectral Scanner (MSS) data combined with radar measurements from the European remote-sensing satellite with synthetic aperture radar (ERS-1 SAR) may be used for obtaining information on land use and crop areas. The temporal normalized differential vegetation index (NDVI) can be used to monitor vegetal cover and crop growth. The low-resolution advanced very high resolution radiometer (AVHRR) satellite imagery can be used operationally to estimate annual crop area, derive 10-day yield indicators and derive quantitative estimates of crop condition and production. Currently, the National Meteorological Services in a number of countries routinely provide NDVI maps, derived from the VIS, or visible, and NIR, or near-infrared, channels, on a monthly basis for monitoring the vegetation growth and for crop forecasting in support of real-time irrigation and drainage management.

There is a great deal of literature on the use of satellite remote-sensing applications in irrigation management (Bastiaanssen, 1998; Musiake and others, 1995; Vidal and Sagardoy, 1995; Kurtas and Norman, 1996).

4.5 HYDROPOWER AND ENERGY-RELATED PROJECTS [HOMS K10, K15, K22, K45]

4.5.1 General

Man has always exploited hydropower. The first record of its application as a mechanical force goes back to Hercules who, in Greek mythology, deviated a river to clean the stables of Augias.

The driving force of water has long been transformed into mechanical force for use in mills and factories. The advent of electricity at the end of the nineteenth century made it possible to transform this hydraulic power into electric power, which is more easily transmitted far from its source for use elsewhere. The use of hydroelectricity rose rapidly during the twentieth century and continues to have a promising future today.

The recovery of the driving force of water is achieved primarily in two ways:
(a) Using the streamflow (speed of the water mass flowing in the riverbed);
(b) Using a drop in hydraulic head, that is, transformation of potential energy into kinetic energy by a change in altitude.

Another energy use of water is its use as a cold source for thermal power stations operated by coal, oil or nuclear fuels. Water is necessary in practically all the technical stages of thermo-electric energy production, from drilling exploratory bores in layers of gas and oil, to the transformation of fossil and nuclear fuels into electric power in thermal power stations.

The fundamental difference between the production of electricity of thermal origin and hydroelectric power stations lies in water consumption. A thermal power station will use water for cooling: part of this water will be evaporated by the energy production system and part will be rejected at a temperature higher than that of the withdrawal. A hydroelectric power station will return the same quantity of water
to the natural environment, minus any loss by evaporation from reservoirs, but generally with a somewhat different hydrological regime.

In river management, a series of various types of power stations along the river must be considered at an early stage because of the potential for conflict among the users regarding their needs. The impacts of these installations are broad: variations in flow, heating of the water, reduction in the number of fish species, evaporation, diversions into or out of the catchment area, risks of pollution and so forth.

4.5.2 Hydropower

Hydropower is renewable energy, derived originally from the sun, which drives the water cycle, causing rivers to flow over millennia. Hydropower uses this energy without consuming water to any great extent and can therefore be described as sustainable energy as defined by the United Nations World Commission on Environment and Development: “...development that meets the needs of the present without compromising the ability of future generations to meet their own needs.”

Hydroelectric schemes are diverse, not only as a result of the different natural conditions to which they may be adapted, but also because of the diversity of circumstances related to power demand and use. Hydroelectric power is frequently developed as part of a multi-purpose project so that the project may involve the full range of water resources considerations, such as flood control, navigation, irrigation, municipal and industrial supplies, recreation, and fish and wildlife enhancement. Further information on multi-purpose projects is available in 4.1.

A project is rarely restricted to a local area. In most instances, it deals with an entire river basin, entailing regional, national and international considerations. In considering any magnitude of development, the planning phase must take into account all water resources needs of the region and the ways in which such needs are to be met. The effects of a hydroelectric development project on the resources and various needs in a region, and its capacity to meet those needs, must be carefully evaluated.

Although hydroelectric projects have become increasingly large during the past century, small hydroelectric plants of up to a few megawatts can economically exploit the energy at potential sites on small streams, or they can often be integrated into existing dams or artificial waterways.

4.5.2.1 Advantages and disadvantages and impact on the environment

4.5.2.1.1 Advantages

Although hydroelectric installations throughout the world meet around 20 per cent of global demand for electrical energy, their output is proportionally greater than that of other sources. They use energy, the supply of which, in almost all countries, is prone to risks associated with climate variability and change, but not to political or economic risks. Hydroelectric energy is especially significant as an economic stimulus in developing countries and as an important part of complex power systems in more industrialized countries. Its importance will not diminish for the following reasons:

(a) It is derived from a continuously renewable resource powered by the energy of the sun;
(b) It is non-polluting – significant heat or noxious or greenhouse gases are not released in its production;
(c) Hydroelectric plant efficiencies can approach 95 per cent, whereas fossil-fuel-fired thermal plants attain efficiencies of only 30 to 40 per cent;
(d) Hydroelectric plants have a long, useful life, if properly maintained;
(e) Hydroelectric technology is a mature technology offering reliable and flexible operation, and its equipment can be readily adapted to site conditions;
(f) Water in storage provides a means of storing energy and may be available for other purposes;
(g) Hydroelectric plants are capable of responding within minutes to changes in electrical demands;
(h) Hydroelectric generation has no fuel costs, and low operating and maintenance costs mean that it is essentially inflation proof;
(i) It replaces the use of fuels which would otherwise have to be imported or, if produced nationally, could be exported, thereby improving a country’s balance of payments;
(j) It generates a source of employment during its construction, exploitation and maintenance, and helps reactivate regional and national economies.

4.5.2.1.2 Disadvantages

Hydroelectric energy does, however, have some disadvantages, as follows:

(a) Capital costs are relatively high;
(b) There is only a limited possibility for a stage-by-stage construction, possibly meeting a
-growing demand for electricity, especially because the largest investment must be made at the beginning of civil engineering works on the river;
(c) Production is often far from the centres of consumption;
(d) Construction of hydroelectric energy plants is a lengthy undertaking;
(e) The rivers and lakes concerned are not private property and the decision to develop hydropower must be taken at the national level, involving thorny political negotiations – planning, construction and return on investment may extend over several decades;
(f) Potential destruction of natural habitats and the loss of plant and animal species.

4.5.2.1.3 Environmental impact

A hydroelectric power installation clearly has an impact on the environment, as described in 4.2.8, and more specifically in 4.2.8.3.

In particular, it can have the following impacts:
(a) A modification of the river's flow regime;
(b) A fill of stored water volumes of one part of the year on another;
(c) Unnaturally rapid variations in streamflow;
(d) Flooding of upstream areas.

It will therefore be necessary to assess the other uses of water upstream and downstream from the planned installation so as to take them into consideration in the design and operation of the installation.

4.5.2.2 Types of installations

It is somewhat difficult to classify hydroelectric installations because they are all unique and they are adapted in each case to a river's geomorphology, its hydraulic regime and the consumption needs of an area or a country as a whole.

The following is an attempt to classify them according to their position on a river and their type of operation:
(a) The upstream part of a river generally has a steep channel (see the hypsometric curves in Figure I.2.21, Volume I, Chapter 2) and highly variable and seasonal low flow. Accordingly, a high-head power plant will generally be installed;
(b) The middle reaches of a river have only a moderate slope, but a steadier flow. This will lead to the installation of a medium-head power plant;
(c) The downstream part of a river will often be broad and feature a shallow slope, but will have a constant flow which suits the installation of low-head or run-of-river power plants;
(d) The last category of installation includes pumped storage plants.

4.5.2.2.1 Power of an installation

Hydroelectric energy is developed by transforming energy in water that falls from a higher level to a lower level into mechanical energy on the turbine generator shaft and into electrical energy through the generator rotor and stator. The power potential of a site in kWh is a function of the discharge and of the head as indicated below, and its exact expression is as follows:

\[ P = 9.81 \eta Q H \]  (in kW)  

where \( \eta \) is plant efficiency, \( Q \) is discharge in m\(^3\) s\(^{-1}\), and \( H \) is the net head (fall) in metres, that is, the total height between the upstream level and the downstream level.

It is therefore necessary to know the exact power that will be produced and to have a clear definition of the project components. Within the framework of a preparatory project and pre-dimensioning, the following formula could be used, giving a very good power approximation:

\[ P = 8.5QH \]  (4.8)

where \( Q \) is the discharge (in m\(^3\) s\(^{-1}\), ) and \( H \) the net head (in metres).

4.5.2.2.2 High-head power plants

These power plants (see Figure II.4.16) are characterized by a specific hydrology because of their location close to the source of the river, often in high mountains with a small catchment area. The
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hydrological regime will be typical of such locations: a highly variable flow according to the season, directly influenced by strong mountain rains and, if at higher altitudes, by the snowmelt. As a result, there are periods of very high flow and periods of very low flow. The dam must therefore be able to store water at the times of high flow so that it can be used when there is a demand for electricity. If the high flows come from snowmelt, the reservoir must be sufficiently large to store all the water held upstream in the snow pack.

The means of calculating the dimensions of the reservoirs are described in 4.2. Because of the steep slope of the river, it can be arranged in such a manner that a large difference in head exists between the reservoir level and the turbines without the need to transport the water long distances. It will then be possible to generate a large amount of power, in spite of comparatively low flows (see equation 4.7). This relation for high dams can be expressed in the following manner:

\[ P = 9.81 \eta QH \]  
(4.9)

Installations of this type can store water and thus transfer it from one season to another and can have a complementary use in maintaining flow rates during periods of low flow to match electricity demand.

4.5.2.2.3 Medium-head power plants

In the middle reaches of a river, the flow is already more regular than upstream and the slope is still sufficient to provide a useful head in the order of 40 to 100 metres. It is therefore possible to install a dam that will allow some of the flow to be stored during times of low electricity consumption: at night, hours of low activity or on non-working days (see Figure II.4.17). Moreover, during periods of high demand, stored water can be released in order to create a flow through the turbines that is above-normal streamflow. This type of operation can operate on a daily, weekly or monthly basis.

The power of such an installation can be presented in the following manner:

\[ P = 9.81 \eta QH \]  
(4.10)

4.5.2.2.4 Run-of-river power plants

This type of installation, also known as a low-head power plant (see Figure II.4.18), provides no water storage for later use and energy production is fully dependent on the current flow in the river. All or a part of the flow passes through the turbines and is returned immediately to the river. There is thus no modification of river flow. If a turbine is stopped because there is no demand for electricity or it needs to be repaired, the flow must be maintained and diverted through an alternative route using valves or a bypass channel. Water thus diverted will be lost for the supply of electricity. Since run-of-river power plants operate permanently, a detailed study of the river regimen is necessary to dimension the turbines and other characteristics of the installation.

Floating mills are increasingly being used on large rivers. These are made up of water wheels, which operate electric alternators. The mills are installed on barges and are positioned on the river using cables and winches. The advantage is that they rise and fall with the level of the river, they can be brought back to the banks for maintenance and during floods and can always be positioned at locations of maximum flow. Moreover, the investment is modest and the small electric generating units that are required can be built locally. There is no need to construct civil engineering works on the river and only the winches need to be anchored on the banks. The great disadvantage is the yield of the paddle wheels, which is only 30 to 50 per cent.

figure II.4.17. Medium-head power plant

figure II.4.18. Low-head power plant
4.5.2.2.5 *Pumped storage power plants*

It is impossible to store large quantities of electricity. Thus the electricity produced at times of weak demand can be used to pump water and to store it in a reservoir at a good height above the river. When demand rises again, it is then possible to release the water through turbines to produce electricity. The total yield of the operation is approximately 70 per cent, but it can be profitable if the energy used for pumping would otherwise have been lost because the low-head turbines on the river would have been stopped. Plants of this kind resemble high-head power plants and often the pumps are reversible and also serve as turbines. The yield may not be high, but it can be important because of its flexibility within the overall generating capacity of a region or country.

4.5.2.3 *Structure of a hydropower plant*

A hydroelectric power plant comprises several structures which are, from upstream to downstream, as follows: the intake, headrace, penstock, powerhouse, tailrace or discharge water passage, and related structures such as fish ladders and a system for providing compensation water. See Figure II.4.19.

4.5.2.3.1 *Intake*

The intake (see Figures II.4.20 and II.4.21) is necessary to divert water from the river and direct it to the turbines. The intake is necessarily located near the riverbed, and is frequently incorporated in a dam.

The principal rules to be observed are as follows:
(a) The position in the river must be so that floating objects do not block the intake;
(b) The intake must be provided with protective grids to prevent objects from entering the turbines and fish from being trapped in the powerhouse. In the latter case, the spacing of the bars must be based on local regulations and may be only a few centimetres;
(c) The surface area of the grids must be such that it allows the flow to pass without creating too great a loss in pressure.

As necessary, a system should be installed to allow the removal of floating objects that frequently accumulate upstream of the intake.

4.5.2.3.2 *Headraces*

The headrace directs water to the power plant, which is often far from the intake, in order to...
benefit from as great as possible a difference in height between the level of the intake and that of the discharge point into the river. They are either open-top or covered canals, or tunnels with either open or closed conduit flow.

Open channel flow is possible only if the headrace leads from the top of the dam; the first part of the headrace depends to a large extent on the topography of the site. In general, the slope of open channels is gentle and the water velocity is limited to about 2 m s\(^{-1}\). Open channels often lead to a pressure pipe, or penstock (see Figure II.4.21), which guides the water from the channel or tunnel to the turbine down a very steep slope. All of these parts are equipped with valves to cut the water flow and to isolate them from the river for inspection and maintenance purposes. Additional works, such as surge tanks or standpipes, are installed to accommodate accidental excessive pressure rises.

Pressure pipes are often made of metal, but can also be made of reinforced concrete, pre-stressed concrete or, as in the past, wooden planks assembled in a barrel-like form.

4.5.2.3.3 **Powerhouse**

The powerhouse is the building or the underground excavation that contains the generating units: the turbines and alternators. It must be adapted to the size of the generating units and, in most cases, should include maintenance or repair shops. In general, electric units, such as voltage transformers and the terminals of the electric cables feeding the network, are coupled with the powerhouse.

4.5.2.3.4 **Tail race or discharge water passage**

Tail races are needed to return the water to the river after it has passed through the turbines. The part which connects the powerhouse to the river depends primarily on the type of turbine and thus on the fall in head. These discharge passages very often include a gate to isolate them from the river in case of an emergency. Where a number of power plants are installed in a series along a river, the tail race of one could be used directly as the intake for the power plant downstream.

4.5.2.3.5 **Related structures**

In hydroelectric power plants, it is often necessary to build structures which are not directly required for the production of electricity production, but are necessary for water management and the observance of regulations. Examples include the following:

(a) Fish ladders (see Figure II.4.23) – to allow migrating fish to bypass dams and not be crushed by the turbines;

(b) Compensation water systems – to return a part of the discharge of the river directly to the foot of the dam. This prevents the reach of the river between the intake and the discharge of the tail race – the short-circuited section – from becoming dry so as to preserve the aquatic life of the river and allow other uses of water along the short-circuited section. In this section it is normal to maintain a permanent flow in the river, the rate being set according

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**Figure II.4.21. Example of a partially buried penstock**

**Figure II.4.22. Plan of a typical powerhouse**
4.5.2.3.6 **Special provisions**

It is becoming increasingly frequent for medium to small power plants to incorporate all these structures within one barrage power station. This has the advantage of decreasing the length of the headraces and reducing the short-circuited section to zero. Such installations can pass a constant flow in the river while continuously generating electricity. However, this is not possible without the construction of a dam and a river geometry that gives rise to a desirable head difference at the selected location.

4.5.2.4 **Power plant flow determination**

A reliable estimate of the energy that can be generated at a selected site depends to a large extent on the type of power station that is going to be constructed and on the hydrology of the upstream river basin. The hydrological study of the river at the power plant location should be as exhaustive as possible, and should include the following information, which makes it possible to determine power plant streamflow requirements:

(a) Daily and monthly streamflow data for an extended period of time – more than 10 years, preferably 30 years, if possible;

(b) Flow-duration curve or flow-frequency curve;

(c) Historical records of floods near the site;

(d) Computed design flood;

(e) Mean annual discharge;

(f) Minimum annual flow;

(g) Minimum-flow requirements downstream from the site;

(h) Streamflow diversions upstream from the dam or intake works;

(i) Drainage areas;

(j) Evaporation losses from proposed reservoir surfaces;

(k) Stage–discharge relationship immediately below proposed site;

(l) Spillway design-flood hydrograph;

(m) Dam, spillway and outlet rating curves;

(n) Project purposes, available storage and potential operating rules;

(o) Seepage losses, fish bypass requirements and other diversions from storage;

(p) Reservoir elevation-duration information;

(q) Annual peak-discharge data to assess risks associated with spillway design.

The flow-frequency curve, or flow-duration curve, illustrated in Figure II.4.24, classifies the daily average flows of an average hydrological year (see Chapter 5) and is widely used at the preparatory project stage. It indicates the number of days in the year, or the annual return frequency, for which a given flow is reached or exceeded, making it possible to estimate the potential production of a hydroelectric power station as a function of streamflow reliability. In turn, an estimate can be made of the profitability of the investment.

By using this approach, different production strategies can be simulated based on the physical design of the dam and the number of turbines installed. In addition, requirements of other demands on the water, such as minimum-flow, irrigation or drinking water supply, can be taken into account.

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**Figure II.4.23. Example of a pool and weir fish ladder, allowing fish to bypass the dam and reach the higher water level through a series of low-head weirs**

**Figure II.4.24. Flow-frequency curve of daily interannual streamflow data***
The recommendations and methods presented in this Guide provide a good basis for studying the hydrological characteristics of an area. In particular, if river flow data are not available, methods for deriving data from rainfall-runoff models or from neighbouring basins may be used, but they introduce additional uncertainty which should be taken into account when estimating the potential production and hence return on investment.

Other techniques used in the analysis of hydrological data to extract relevant information for design purposes are given in Chapter 6. The Hydrological Operational Multipurpose System (HOMS) Reference Manual (see Section K) provides information on the availability of software packages for the application of these techniques.

**4.5.2.5 Determination of the head**

The head of a hydroelectric plant is determined by the geographical and topographic characteristics of the river. It is important to distinguish between the following factors:

(a) The gross head, which is the difference between the water level upstream of the intake and the water level in the river at the point of discharge downstream or, in certain cases, such as when partially submerged runner-wheel turbines are involved, at the wheel axle;

(b) The net head, which is the pressure of the water as it flows into the turbine. It is derived by subtracting from the gross head any energy losses, including the losses of potential energy due to friction caused by components such as grids, valves and pipelines. These energy losses are a function of water velocity in the inlets and can reach several tens of metres where high heads are concerned.

Therefore, the net head, which determines the power of the machine, varies constantly. It depends on the following parameters:

(a) The surface level of upstream water, which can differ according to the season for many power plants and dams;

(b) The flow used by the turbines, which varies according to the demand for electricity;

(c) The river discharge or flow, which can raise the level downstream, for example during floods when the spillway passes large flows. This happens even in low-head power plants, where there may not be enough gross head to make the turbines rotate during floods.

The net head is necessarily lower than the gross head, and it can only be calculated once the
engineers have decided on the actual elements to be installed. The proposed preparatory project formula (see equation 4.8) considers only the average energy losses observed on a large number of power plants. For a precise calculation of the power produced, it is often necessary to simulate the factory’s operation on a daily basis throughout the year and determine the average net head.

Therefore, the power of a hydroelectric power plant varies significantly according to the prevailing hydraulic conditions but, by convention, the maximum capacity of a power plant is always given as the power generated at maximum flow under the largest net head.

4.5.2.6 Production of a generating plant

The energy produced by a hydroelectric power plant is the most important factor to determine because it makes it possible to estimate the annual income of the system and hence its financial viability. Production is deduced from the power in kWh:

\[ E = \sum P_t t_i \]  

(4.11)

where \( E \) is the energy in kWh and \( P_t \) is the power of the plant during the time period \( t_i \).

The power results from the net head at a given moment with the river discharge at the same moment. This calculation can be made using the mean net head associated with the mean flow over a period which is hydrologically relatively homogeneous.

To calculate the income, it is necessary not only to evaluate the quantity of electric power produced, but also its production schedule as related to the selling prices of electricity because the price varies on the markets.

It is therefore advisable to consider setting fixed prices for certain periods in the concerned area and to carry out a simulation of production according to each price period in accordance with the usable flow at the same periods. Although done by economists, the survey is based on the outputs of forecasting models developed by hydrologists.

In preparatory project studies, and in areas where the number of hydropower stations to be considered is limited, production over a given period can be estimated by using the following formula:

\[ E = \frac{8AH}{3600} \]  

(4.12)

where \( E \) is the energy in kWh produced during a certain period, \( A \) is the volume of usable water in m\(^3\) during the chosen period and \( H \) is the gross head in metres.

This simplified formula takes into account the average energy losses of power plants, as well as the mean efficiency of all elements installed. It generally gives a precision of about 5 per cent.

4.5.2.7 Water quality

Water quality is generally not a major concern in hydroelectric projects, although they can have an effect on it. Various studies and recent experiments show that certain lakes can become eutrophic, either because they have been used as a recipient of urban or industrial wastewater, or because, at the time of dam construction, the flooded zone was neither cleaned nor deforested. The decaying vegetation can cause a significant reduction in dissolved oxygen, severely limiting aquatic life for many years. This can be a very important consideration if pisciculture or other water activities are practised in the area of the establishment.

As a result, water may become so acidic and corrosive that it may attack the runner blades and other parts of the turbine machinery (see 4.9, in particular 4.9.2.2). A more serious effect can be the discharge of de-oxygenated water into the river downstream, which can destroy fauna and flora several kilometres deep, bringing all fishing activity to a halt.

Another risk is related to sedimentation in the reservoirs: sediments brought into the reservoir by the river settle to the bottom because of the low speed of water and then undergo decantation at the bottom of the reservoir. These sediments can contain pollutants such as heavy metals – lead, arsenic, copper – which concentrate in the reservoir and can reach dangerous levels.

In certain cases, by draining the reservoir under flooding conditions, sediments can be cleaned out and returned to the river. However, such an operation should be studied with care to ensure that it does not pose a threat to the downstream reaches of the river.

4.5.2.8 Hydroelectric project stages

When constructing a hydroelectric power plant, it is essential to proceed on the basis of a clear plan in order not to omit any important details, and to correctly evaluate the profitability of the
investments which will be substantial, especially if a dam is necessary. Small systems, for example a run-of-river power plant with a simple dam, may have a financial amortization period of 8 to 10 years, compared with 30 to 50 years for large systems. Thus the quality of the study depends on two major criteria:
(a) The hydrological study, on the basis of which the potential output and annual risks are assessed;
(b) The geophysical study, which is used to locate the best site in order to have the greatest possible head in the selected zone.

4.5.2.8.1 Hydrological study

As previously stated, this study must be as complete as possible, outlining the methods used to determine the flows and other characteristics so that the uncertainty related to the evaluation can be assessed. It is necessary to have daily or, at a very minimum, weekly outputs. It is also essential to know the risk involved in using these averages because they can mask a flood that might be too massive to pass through the turbines and will have to pass through the spillways. Indeed a monthly flow of 100 m$^3$ s$^{-1}$ might be the result of a mean flow of 30 m$^3$ s$^{-1}$ for 28 days and a flow of 1 080 m$^3$ s$^{-1}$ for two days, or even of a slowly varying flow ranging from 120 m$^3$ s$^{-1}$ to 80 m$^3$ s$^{-1}$. The difference will have a major impact on the energy that can be generated in a month.

It is important to conduct a careful study of flood frequency in order to dimension the works that will be needed to handle such high flows without damage to the dam or power plant. It is also necessary to evaluate the project flood, which is the maximum flood that will be passed without any damage to the work; a larger flood will be likely to cause serious damage to the installations. In many countries project floods are defined in regulations and computed according to downstream risks.

In carrying out a flood study, it will not only be necessary to calculate the flow to dimension the spillways, but to locate all the high-tension electrical installations, including the power station itself. Unfortunately, as a result of inadequate flood studies, power plants are sometimes submerged, and electrical installations destroyed, by floods of a frequency of only a few tens of years.

Finally, to be complete, a hydrological study must consider the various uses of water and how the project will make it possible to respect them. It must also take into account the various problems of flow related to existing installations such as bridges, mills, dams and fords in the reservoir’s zone of influence and downstream of the power plant. Clearly, as the project takes shape, an increasing amount of information must be provided. This will require the expenditure of significant funds at the preparatory project stage. If these funds are not made available and an adequate hydrological study is not undertaken, the profitability of the whole project will be at stake, with a likely loss of major investments at stake.

When computing the necessary hydrological elements, it is important to remember the flows that will have to be assigned to other water users and to determine jointly whether the flows can pass through the turbines and thus generate electricity. If so, it must be decided when and how, or whether they must be diverted upstream of the headrace. The outcome of these studies can change considerably the project’s financial viability. In general, these studies involve regional or national governments.

4.5.2.8.2 Geophysical study

Along with the hydrological study, it is necessary to obtain as much information as possible to evaluate the potential head. Plans, existing surveys or medium-scale maps are generally used to carry out geophysical studies, but should be supplemented by field surveys. The information collected in the field is essential to determine the position of the dam and of the intake, to choose the best location for the power plant, and decide on the most economic means of connecting the two.

Such field visits are also vital for locating traces of old floods, identifying different uses of water in the area and determining whether the potential location of the reservoir includes zones where water might be lost through infiltration or zones where fauna or flora will need to preserved.

4.5.3 Operation of a hydroelectric system

The operation of a hydroelectric system is very complex. It is defined by its generation capacity and the demand for power supply. It is necessary to find a balance between present and future power generation, because the generation of large amounts of electricity in the present can lead to a energy-production deficit in the future. However, a low level of generation in the present may lead to the excess storage of water, which will need to be released later. Therefore, it is necessary to use a design procedure that will optimize the use of water,
maximizing benefits and minimizing costs. See 4.2 for further information.

4.5.4 Other projects related to energy production

As stated at the start of this section, while the principal use of water in the generation of electric energy is through hydroelectricity, water is also essential in the production of thermal energy. A guide to the quantity and quality of water necessary for various thermoelectric energy-generating processes is provided below.

4.5.4.1 Production of energy from fossil or nuclear fuels

The use of water in the production of electricity is identical to fossil or nuclear fuels. All thermal powerhouses use water for the production of vapour and for the cooling system, and, to a lesser extent, for general services such as for drinking. Rivers and lakes serve as the cold source necessary for the Carnot cycle.

The volume used depends essentially on the characteristics of the system used for cooling-condensation and evacuation of heat. Water as a coolant in the condenser is the most important use and the necessary quantity for this purpose is in the order of 0.032 to 0.044 m$^3 \cdot s^{-1} \cdot MW^{-1}$ on the basis of an increase of temperature of 8°C. The principal means of dissipating the residual heat are dry cooling towers and the direct discharge into rivers of the effluents from the heat exchanger. The application of regulations designed to limit excessive warming of rivers has resulted in a reduction in the use of the direct discharge into rivers. Evaporation cooling towers are the largest water consumers, discharging only the condensed water into the river. Dry cooling towers disperse the residual heat of the plant directly into the atmosphere by means of thermal exchangers cooled by air, without the addition of heat to the natural water bodies, and without their consumptive use. Thus, the plants that use this system need a larger amount of fuel and an additional plant investment.

In the case of power plants using coal dust as fuel, water is also needed to transport the ashes. This demands about 0.00095 m$^3 \cdot s^{-1} \cdot MW^{-1}$, and desulphurization of the combustion gasses with a demand of about 0.0000019 m$^3 \cdot s^{-1} \cdot MW^{-1}$.

As in any other complex system, nuclear plants are exposed to numerous unpredictable problems that can interfere with their normal functioning and in extreme cases can endanger the health and security of the population. The possible occurrence of serious accidents is undoubtedly very low because severe safety and safeguard measures are implicit in nuclear plant design (IAEA, 1981). WMO (1981) describes the different types of nuclear power plant and analyses the problems connected with hydrology and water resources that should be considered in the planning, design, exploitation and the shut-down of nuclear plants.

This publication contains some examples of the technologies used to address important questions at varying levels of complexity. Both high and low flows are of special importance for the management and security of a nuclear plant. It is essential that emergency cooling of the nuclear core, the cooling of the used fuel and the final heat sink have a reliable water supply. Protection against flooding is also very important, regardless of the type of power plant, because it can interrupt normal operations and especially if it affects two or several systems, thereby reducing the effectiveness of emergency safety systems. Therefore, it is generally necessary to apply the best available hydrological forecasting system to the basin upstream of a power plant and to perform periodic revisions of the hydrological analyses and assumptions made in the planning of the plant.

In most energy-related projects, considerations relative to water quality do not control the feasibility of the project, but they can influence its size, the type of procedures used, the choice of location and other factors. The composition of groundwater from different sources varies considerably in terms of dissolved salts and gases. Surface waters generally contain suspended load and often, dissolved or suspended organic matter, originating from rotting vegetation or from wastewater. The growing use of synthetic detergents, some of which cannot easily be destroyed in wastewater treatment processes, results in the presence of measurable quantities of these chemicals, even in drinking water supply reservoirs.

Rainwater can have a low pH and be potentially corrosive in industrial zones because of coal dust and oil particles. If these are carried by the wind, the impact can be significant even at a great distance from the emission sources. Most waters, however, can be treated to be used in the cooling by condensers, transport of ash and desulphurization of combustion gasses. Nevertheless, boiler feed requires pure water, without any trace of dissolved salts. The cost of preparing pure water generally
increases with the quantity of salts dissolved in the water.

To a large extent, radioactive waste from nuclear power plants is caused by events such as leakage, blowdown, maintenance and fuel restocking. The water that circulates through the reactor is used as a heat source and the products of the corrosion created in the system are the principal source of radioactive isotopes in water in the reactor. It is essential that the water used for cooling and feed water be exceptionally pure, as all salts or other impurities contained in the water can capture neutrons and make them radioactive. The products of fission within the fuel elements constitute another potential source of radioactive isotopes in the reactor water. Therefore, the quantity of radioactive isotopes present in the reactor water depends on the rate of corrosion, any failure in the coating of fuel elements and its rate of elimination by condensation or by cleaning of the reactor. The possible presence of radioactive isotopes in water calls for special precautions to be taken in waste treatment.

In the primary circulation system, great attention should be given to maintaining water at a high standard of purity in order to minimize the accumulation of excessive radioactivity due to impurities or corrosion products. There may be no loss of primary water, but some of it is extracted, purified and recycled. The possible danger of corrosion under pressure implies that the boiler water contains very low concentrations of oxygen and chlorides. To reach this level of purity, the water used in the primary circulation system must undergo deaeration and evaporation treatments so as to reduce the levels of oxygen and chloride to less than 0.003 and 0.3 mg/l, respectively.

4.5.4.2 Coal extraction

The extraction of coal, whether from open or underground mines, uses only a small quantity of water. In fact, the infiltration of water underground can be an obstacle to mining activity and may require considerable effort and investment to remove it. The production of coal dust makes use of large quantities of water for washing the dust but recycling systems are generally employed.

Activated carbon sludge technology has been used since the beginning of the twentieth century. The transport of coal sludge can be economical for high volumes or over long distances, but after the separation of the pulverized coal dust, the water must be treated before being discharged into a natural watercourse. Wastewater treatment facilities will depend on the quality of the coal dust to be transported – its content in sulphur, ashes and minerals – on the chemical additives necessary to inhibit corrosion in pipes and equipment, as well as on the chemicals used as coagulation agents in the procedure.

Wastewater from coal mines sites contain a wide range of metals, suspended solids and sulphates originating from pyrites and/or from marcasite, which are commonly associated with coal deposits, and from schist and sandstone. If exposed to the atmosphere, these minerals form sulphuric acid and compounds of ferric hydroxide. Whether in settling ponds, slag heap of waste rock or wherever it is stored, coal dust can therefore produce acid drainage. The impact on the receiving waters will be to produce a high degree of acidity (pH of 2 to 4) and high concentrations of aluminium, sulphate, iron and trace amounts of heavy metals.

4.5.4.3 Uranium extraction

Little water is used in underground or open uranium mines and what is used is mainly as drinking water. The total use of water during the tertiary crushing of uranium is also small and it is mainly used to lubricate the crusher.

Uranium concentration generates both radioactive and non-radioactive waste and effluents. Solid, liquid and gaseous effluents may be discharged into the environment in large or small amounts, according to the procedure in place to check and control the release of the waste.

4.5.4.4 Petroleum production

Water supply availability and cost, together with energy conservation and environmental concerns, have an impact on petroleum processing. Modern refineries are designed so as to reduce water demand to some two per cent of what it was for the older refinery systems and procedures. Currently, great importance is given to air-cooling in place of water-cooling and to the multiple uses of water, including recycling. The level of water utilization therefore depends on the age of the refinery and tends to be directly proportional to the capacity and complexity of the refineries. The demand for water can fluctuate between 0.1 and 3 m^3 bbl^{-1} according to the size and complexity, and the processes used by the refinery.

Effluents from petroleum production and the refining process need to be treated before being released into natural watercourses. Such treatment mostly
involves the use of settling tanks and the separation of petroleum from water. Because of the high quantities of water required by some procedures, recycling has become necessary in new refineries.

4.5.4.5 Methanol production

The conversion efficiency for producing methanol fuel from wood or natural gas is approximately 60 per cent. Therefore, a large proportion of the heat content of the original carbon-rich source materials must be rejected during the process. Approximately half of the heat loss can be rejected via an evaporation cooler, requiring approximately 3 m$^3$ of water to be evaporated for every tonne of methanol produced. Alternatively, if direct cooling is used, and a 10°C temperature rise is permitted, 170 m$^3$ of water would be passed through the heat exchanger to remove this heat with an induced evaporation loss of 1.5 m$^3$/tonne of product. Clearly, if water is scarce or costly, the design must include a means of eliminating heat that is efficient in its use of water.

4.6 NAVIGATION AND RIVER TRAINING

4.6.1 Application of hydrology to navigation

Rivers are characteristic landscape features and part of the natural, cultural and economic environment. Besides their function as navigable waterways, they have great significance in terms of the national economy and ecology.

During the early developmental stages of navigation, transport facilities were primarily dependent on the characteristics of the rivers or river reaches concerned. Over time, the need for increased transportation capacity led to the development of uniform navigation conditions by means of river canalization or river training, which allowed long-distance transport on ever larger ships without frequent and expensive transhipment.

Since the early times of river navigation, depth and width have been the basic parameters of waterways. There are different concepts of waterway development. According to classical river-regime theory, river engineering based on hydrological characteristics is preferred when dealing with free-flowing and strongly meandering flatland rivers, while hydraulic engineering is the method of choice where steeper river reaches, including those with reinforced embankments, are concerned. The number of parameters that can be taken into account depends solely on the computer capacity available. Increasing emphasis is now being placed on the interaction between ship design – form, draught, mode of propulsion – and the structure and routing of waterways. As regards hydrological features, a general differentiation must be made between free-flowing and impounded or canalized river reaches or artificial canals. Hydrological–hydraulic parameters and the features of the interaction between ships and the waterway characterize and define the quality of any navigable waterway.

Some factors that influence navigation remain more or less constant over long periods and can be described by well-defined parameters. Other factors, however, characterize the temporally variable navigation conditions that depend on the streamflow regime of the river, particularly on events such as floods and low-flow periods. An example of an event with negative consequences was the prolonged low-flow period in the Rhine river in August 2003 (see Figure II.4.26). Another key factor is the upstream catchment of the river: its type and size and flow over the course of the year.

Hydrology plays a key role in two primary aspects of river navigation:
(a) The characterization of river reaches with respect to the types of vessels that regularly use them for navigation (for example, waterway classification according to Figure II.4.27);
(b) The current hydrological conditions that control the operation of vessels as a function of navigable depth or equivalent water levels, for example.

Figure II.4.26. Low flow in the Rhine river in August 2003 hinders navigation
CLASSIFICATION OF EUROPEAN INLAND WATERWAYS

<table>
<thead>
<tr>
<th>Type of inland waterway</th>
<th>Class of navigable waterway</th>
<th>Main vessels and barges</th>
<th>Padded vessels</th>
<th>Minimum length under bidding</th>
<th>Graphical representation on maps</th>
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<td>Maximum beam</td>
<td>Length</td>
<td>Beam</td>
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<td>Length</td>
<td>Beam</td>
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<td>L00</td>
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<td>I</td>
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<td>5.55</td>
<td>1.05-2.20</td>
<td>155-400</td>
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<tr>
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<td>2.50</td>
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<td>8.2</td>
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<td>155-1,000</td>
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<td>195.298</td>
<td>32.2</td>
<td>1.60</td>
<td>18,000</td>
</tr>
</tbody>
</table>

Figure II.4.27. Example of classification of waterways

These two aspects are discussed in greater detail in the following sections.

4.6.1.1 Application of hydrological data to the characterization of waterways

The goal of an investigation of any waterway in relation to its potential for navigation is to determine the seasonal probabilities of navigability for various categories of vessels on the individual reaches of the waterway. This may be accomplished, for instance, by using a system of categories that are based on relevant parameters, as those defined by the United Nations Economic Commission for Europe. The definitions of several of these parameters are given below:

Waterway or fairway – The part of the river that is passable by ships and ship caravans, marked by means of navigation signs (buoyage).

Navigation clearance – The complexity of factors characterizing the depth, width, height and sinuosity of the waterway required for regular and safe navigation by vessels of given dimensions (see Figure II.4.28).

Minimum depth of waterway (h) – The minimum depth at the navigable low stage that ensures the required width of the waterway.

![Figure II.4.28. Geometrical elements of a waterway](image_url)

Note: See definitions, 4.6.1.1.
Minimum width of waterway (B) – The minimum width at the navigable low stage that ensures the required depth of the waterway.

Prescribed vertical clearance (H) – The minimum vertical difference across the entire width of the waterway between the lower edge of any structure, for example a bridge, and the navigable high stage.

Minimum sinuosity radius (R) – The prescribed lower limit of the sinuosity radius of a river bend measured to the axis of the waterway during navigable low stage.

Navigable low stage (HK) – The critical stage ensuring the prescribed value of water depth and width.

Navigable high stage (HN) – The highest critical stage generally ensuring the prescribed clearance.

Navigation water demand – The streamflow that is needed to ensure the depth required for safety and ease of navigation.

Minimum streamflow for navigation – The streamflow ensuring the navigable low stage in a given cross-section.

Maximum streamflow for navigation – The streamflow ensuring the navigable high stage in a given cross-section.

Navigation season – The part of the year during which navigation is not hampered by ice.

Ford – The transition reach with small depth between two bends of a river (as used in this context).

Principal shallow – The shallowest section along a given navigation reach.

The procedures for describing these parameters are explained in further detail below.

4.6.1.1 Geometric parameters

The determination of the depth and width available for navigation requires a closely spaced series of observation cross-sections (for example echo sounder measurements) along the river. The minimum stage at which the minimum navigable width is still available has to be identified for each cross-section. The navigable low stage for each cross-section is determined by adding the minimum navigation depth, as recommended for the given river, to its minimum stage. The sinuosity radius can be determined graphically from a contour map of appropriate scale with sufficient accuracy.

In order to investigate the possibility of navigation on a river, it is necessary to carry out the above-mentioned procedure for several values of minimum navigation width so that the navigation category of the natural river or waterway classification can be specified.

4.6.1.1.2 Hydrological parameters

In order to determine the degree to which the runoff regime corresponds to the navigable low stage, it is necessary to compute flow hydrographs and duration curves of water stages or flow discharges at defined cross-sections.

The flow hydrographs should, if possible, be determined from daily data of a time series with a minimum length of 30 years and a wide range of probabilities (see Figure II.4.29). In addition, they should be computed for a number of probabilities of exceedance. The periods during which the prescribed minimum depth of waterway is expected with a given probability can be determined by superimposing the levels of navigable low stage on these curves. The durations of these
periods can then be obtained by computing durations and/or probabilities. As the homogeneity of stage data is not always guaranteed, the duration of the relevant discharges should be determined first and then converted to stage data by means of a valid stage-discharge relationship (see Volume I, Chapter 5). It is possible to find the minimum duration of the navigable low stage along the given river reach by comparing the navigable low stage with the flow hydrographs in various cross-sections. For example, according to investigations carried out on the Danube river, the navigable low stage corresponds to the water stage of 94 per cent duration, as computed for the series of ice-free stage data (Figure II.4.30).

In temperate and Arctic climatic zones, the length of the navigation season is primarily determined by the ice regimes of the rivers. On the basis of observed data of the various ice phenomena such as ice drift, complete freezing, ice break-up, and ice cessation (Volume II, Chapter 6), the values of the various phenomena expected with given probabilities can be computed, and the durations of forced interruptions of navigation by river ice can be estimated. The results of such a calculation for the Hungarian reach of the Danube river are shown in Figure II.4.31.

In order to ensure the efficient operation of ice-breakers (breaking, pitching, shredding), it is necessary to obtain and analyse time series of observations of ice thickness. Here it is particularly important to identify the times when it is worthwhile to continue or commence ice-breaking so as to keep the fairway clear of ice, and when such efforts should be abandoned as uneconomical. These times depend heavily on the meteorological conditions controlling the formation and break-up of ice.

4.6.1.3  Hydraulic parameters

The investigation of the flow regime, as described in the foregoing subsection, can only be carried out for selected, relatively stable cross-sections. Therefore it is necessary to estimate the navigable
low and high stages by interpolation for the river reaches between these cross-sections. The most reliable method of interpolation, especially in the case of the navigable low stage, is the development of water-level profiles. This requires knowledge of hydraulic parameters such as the slopes and the roughness of the various river reaches concerned (see 6.3.6).

4.6.1.2 Application of hydrological data to operational navigation

Inland navigation is a complex economic activity that is highly dependent on natural factors. Without reliable knowledge on the state of the riverbed, the streamflow, the ice regime, and their expected variability over time, and the planning and operation of navigation activities would be seriously hampered. In order to provide this information, it is necessary to continuously collect data on the hydrological regime, predict expected changes and transfer regularly these data and forecasts to potential users. In many cases, this is still done in the conventional manner with the support of National Hydrological Services. Recently, however, the modelling systems and information services have become more and more routine and are often used directly by the navigation services themselves, for example the use of the Electronic Waterway Information System on the Rhine river.

4.6.1.2.1 Data collection

Navigation utilizes a wide range of data collected by Hydrological Services. These include:

(a) Data collected on the river basin, such as topography, vegetation, land use and precipitation. This is done in close cooperation with National Meteorological Services and regional planning authorities;

(b) Data collected at gauging stations: stage, streamflow, water temperature, air temperature, suspended-sediment load, bed load and ice phenomena and so forth;

(c) Physiographic data collected along river reaches, such as variations in the river-course, bed structures, fords and their depths, flow direction and velocity, water-surface profiles and ice phenomena.

For most of the data required for navigation, the observation methods are those used in general practice (see Volume I, Chapter 2), although differences arise primarily in connection with measurements made at gauging stations and observations carried out along the river sections between gauging stations.

The transitions between river bends of opposing orientation often contain shallow sections which constitute the most critical points in the longitudinal course of natural waterways. Accordingly, depth measurements of these shallows should be conducted frequently whenever the water depth above the shallow does not reach the prescribed value. The depths should be measured along the crest of the shallow section. As a result of these measurements, the navigable width of the waterway may be determined for the shallow river reach. The length of the river reach in which the water depth is less than the minimum navigable depth should be marked.

Knowledge of the direction and velocity of flow is required to enable reliable manoeuvres of barge trains through critical reaches such as shallow reaches, inlets and outlets, as well as in the headwaters and tailwaters of ship locks. The surface velocity is measured by means of floats, while the direction and velocity of currents within the water body are measured by current metres equipped with direction finders. The latest technique uses the acoustic doppler current profiler principle that makes it possible to measure or compute all parameters needed at any point of a cross-profile.

Standard ice observations made as part of routine programmes at the gauging cross-sections are not sufficient for safe flood discharge and navigation. They must be complemented with respect to the places where observations are made and the phenomena that are observed. The observations must be extended to river reaches between the gauging stations so that an observation point is established at least at every 5 to 10 kilometres. The most important task is to observe river reaches regularly, particularly for ice jams. During periods of drifting ice and at times of freeze-over and break-up, observations should be made daily, while during the period of fixed ice cover and unchanged flow regime, observations made every 5 to 10 days may be satisfactory. The reliability of ground observations may be enhanced and supplemented considerably by aerial surveys and photos. It is recommended that ice maps be drawn at least every 5 to 10 days and disseminated among the competent authorities and users.

Ice predictions for navigation require observations of the first crystallized formations, and then the development of brink ice. Where hydraulic conditions support the forming of frazzle ice, its density should be characterized according to the following three steps: 0–33 per cent, 34–67 per cent and
68–100 per cent of the depth of the river. The density of drifting ice is characterized according to the percentage of the surface area of the river that it covers 0–10 per cent, 11–20 per cent, and so forth, up to 91–100 per cent.

4.6.1.2.2 Forecasting

The efficiency and safety of inland navigation depend on the reliability of hydrological data and of the forecasts of water stages under low-flow and flood-flow conditions, ice phenomena and water depths at narrows and shallow sections. There is a need for both short- and long-term forecasts. Those responsible for navigation are naturally interested in forecasts of flow rates along the navigable stretches of rivers.

In addition to the general methods of hydrological forecasting (see Chapter 7), navigation often uses monthly forecasts that are compiled by taking into account the water volume stored in the river network, both surface water and groundwater. Because navigation is particularly sensitive to the reliability of stage forecasts during low-flow periods, the confidence bands of the forecasts should be narrow. For example, the following values are applied for the Danube river:

<table>
<thead>
<tr>
<th>Probability of exceedance</th>
<th>Width of confidence band</th>
</tr>
</thead>
<tbody>
<tr>
<td>60–70%</td>
<td>50 cm</td>
</tr>
<tr>
<td>70–80%</td>
<td>40 cm</td>
</tr>
<tr>
<td>80–100%</td>
<td>30 cm</td>
</tr>
</tbody>
</table>

4.6.1.2.3 Transmission of data and forecasts

The data collected along a navigable river and the forecasts based thereon can only be utilized if they reach the navigation companies, the shipmasters and the waterways administration in a timely manner.

To ensure this, a well-organized system for the collection and transmission of information is indispensable. For instance, in Germany, use is made of the nautical information radio, or NIF. Such a system is of particular importance on international rivers such as the Danube, which flows through eight countries. In conformity with the recommendations of the Danube Commission, the data collected in the Danube Basin are transferred daily by telex. In order to avoid errors, internationally agreed codes (see Volume I, Chapter 2) have been adopted for data transfer. Announcements reach the shipmasters partly by radio and partly in the form of daily hydrological bulletins.

4.6.1.3 Navigation on lakes and canals

Navigation on lakes and canals differs considerably from navigation on rivers:

(a) The importance of the physiographic and hydrological regimes for ensuring navigation conditions is considerably lower because control structures provide stability of these conditions;
(b) On lakes and impoundments, the duration of ice cover is longer and hence the navigation season becomes shorter;
(c) While problems due to shallows are reduced or fully eliminated, problems caused by silting at heads of reservoirs or ship locks and in harbour basins can arise locally;
(d) Wind impact on navigation increases on lakes and impoundments;
(e) There is a greater dependence of navigation operations on the operation rules of locks and other structures.

The safety of navigation on lakes and canals requires an expanded range of observations:

(a) On the shores of lakes and river impoundments, wind-measuring stations and warning facilities should be established and operated;
(b) In order to minimize siltation by technical means, the amounts of sediment entering and leaving impoundments should be measured systematically to yield a sediment balance;
(c) As barrages create favourable conditions for frazzle-ice formation, regular observations should be carried out in the vicinity of these structures;
(d) Automated stage recorders should be installed at the cross-sections that are particularly difficult for navigation, for example weirs, inlets, and outlets.

In order to be useful, these data must be checked for plausibility and documented, and should be sent to users, such as shipmasters, in a timely manner.

4.6.2 Classification of river training

River training, river regulation and waterway maintenance are continuous activities aiming to facilitate navigation, protect riverbanks and riparian dwellers, and support flood control. Rivers in their natural state often change their beds and, in doing so, cause degradation of the channel and hinder navigation. The discharge of ice and floods show a differentiated picture in this case, depending heavily on riparian land uses and the availability of open land. River training strives to make the river form its own bed with reasonably constant geometrical and
hydraulic conditions, but it also produces a number of undesirable consequences of a socio-economic and ecological nature.

Depending on the purpose to be served, river-training works may be classified as high-water training, low-water training and mean-water training.

High-water training, also known as flood-bed regulation and training for discharge, is aimed at the rapid discharge of maximum floods. It is mainly concerned with the most suitable alignment and height of marginal embankments for the discharge of floods and may also include other schemes of channel improvement for the same purpose. Land-use regulations governing flood plains have essentially the same goal as locally restricted flood control measures, namely the discharge of floods without significant damage or loss of life.

Low-water training is designed to provide minimum water depth for navigation during the low-water season. This is achieved by contracting the width of the channel at low water and is generally carried out with groynes. Low-water training is also known as training for depth.

Mean-water training or mean-bed regulation is the most important of all. Any effort to alter the river cross-section and alignment must be designed in accordance with that stage of the river at which the maximum movement of sediment takes place over a period of a year or more. Although high stages of flow lead to maximum bed activity, such stages are maintained for a short duration; however, there is little movement of sediment at the lower stages that persist for a large percentage of time. In between the two, there is a stage at which the combined effect of forces causing sediment movement and the time for which such forces are maintained is at a maximum. This stage, somewhere near the mean water level, is the most important with regard to influencing the configuration of the river. Mean-water training is concerned with the efficient movement of the sediment load of the river and may therefore be called training for sediment. Mean-water training establishes the basis on which the former two are to be planned (Singh, 1989).

Most commonly used river-training works include guide banks, groynes or spurs and studs, cut-offs, revetments, vegetative protection, gabions and walls.

Figure II.4.32 offers a schematic overview of different aspects of river morphology with morphodynamic processes, including boundary conditions, which show influencing factors and physical processes.

In addition to hydrological data, a great number of other physical, geographical, morphological, meteorological and hydraulic data and/or relationships are required for the design and success of river-training measures. The scope of this Guide does not permit a detailed explanation of many of these variables. Here only the aspects with special relevance to hydrology are discussed.

4.6.3 Erosive forces due to channel flow

In a wide, straight channel exhibiting two-dimensional uniform flow, the shear stress (τ) on the bed caused by the flow is given by the following equation:

\[ \tau = \gamma dS \]  

(4.13)

where \( \gamma \) is the specific weight of water, \( d \) the flow depth and \( S \) the water surface slope (see 4.8). Under uniform flow conditions, flow depth can be determined by a flow resistance equation, such as Colebrook–White’s or Manning’s. The energy slope and water surface slope are equal to the bed slope. This in turn is generally fixed by topographical controls. Bed shear stress thus varies with flow depth and channel gradient and reaches a maximum at peak discharge.

In a straight channel of finite width, the flow pattern and related velocity distribution are affected by bank friction and the boundary shear stress varies accordingly. Maximum shear stress occurs below the maximum velocity filament and subsidiary peaks in the zone of down-welling near the banks. The maximum shear stress on the sloping banks is generally about 0.8 \( \gamma dS \).

Rivers tend to have three-dimensional flow patterns, owing to local variations in channel cross-section, such as pools and riffles, and plan geometry. The related secondary flow distorts the main velocity and shear stress distributions. In this case, equation 4.13 gives only the approximate average shear stress.

The secondary flow in a meandering channel causes peak shear stresses at the base of the outer bank of the bend. Measurements of shear stress distributions in meander bends suggest that the ratio of maximum to average shear stress is a function of the ratio of channel width to curvature, bank roughness and the presence of meander bends upstream (Apmann, 1972). Maximum shear stress values can be up to
three times the average in the upstream approach. Clearly, high local shear stresses at meander bends produce corresponding bank erosion and bed scour.

Any localized feature, such as a bridge or weir, can adversely affect the general flow pattern and cause localized erosion (Neill, 1973). The effects may modify the velocity distribution to introduce a three-dimensional flow field or may increase turbulence. Highly localized erosive activity is likely to remain downstream of the feature until the flow pattern has readjusted to that of the channel.

In general, it is not yet possible to model numerically the boundary shear stress in strongly three-dimensional flows; neither is it generally feasible to take field measurements of design conditions. Therefore, physical models are commonly used to investigate the flow pattern and design parameters (CIWEM, 1989).

Figure II.4.32. Aspects of river morphology – a schematic overview
(German Federal Institute of Hydrology)

Short description: Reshaping of the river bed through current action under consideration of the bed geometry, bottom substrate and sediment yield, including impacts through anthropogenic interventions and navigation

Sub-areas:

- **A** Geometric structures
  - Riverbed
  - Flood plains
- **B** Bottom substrate
  - Composition
  - Structure
- **C** Sediment transport
  - Bed load
  - Suspended load
- **D** Morphodynamic processes
  - Interactions through:
  - Changes through relocation of material, erosion and aggregation
  - Caused and influenced by:
  - Boundary conditions
    - Precipitation, temperature
    - Runoff hydrographs
    - Geology
    - Landscape or vegetation
    - Type of water body
    - Degree of river training
  - Influencing factors
    - Constructive interventions
    - Maintenance activities
    - Dredging, dredged material dumping
    - Navigation
    - Land use
    - Water resources management
  - Physical processes
    - Flow velocity
    - Bottom shear stress
    - Turbulences
    - Flow dynamics
    - Secondary currents
4.6.4 Erosive forces caused by waves and craft

Wave action sets up an unsteady flow field at the bank, which can cause erosion through a combination of the following factors:
(a) Shear stresses caused by run-up and down-rush;
(b) Direct impact of flow onto the bank;
(c) Related seepage flow response in the bank to unsteady external boundary conditions.

The water motion produced by a boat depends on the size and geometry of the waterway, and the boat’s shape, size, speed and sailing line. The components of water motion can be divided into primary and secondary waves and the screw race. The effect of water level drawdown, together with waves and the return current, can cause serious bank erosion, particularly if the blockage factor is high. On a sloping bank, this often manifests itself as a characteristic S shape in the bank profile at around water level (CIWEM, 1989).

In general, the erosive action of the screw race is minor compared with the above effects when the craft is underway, but serious erosion can be caused when a craft is manoeuvring close to the bank or starting off. Velocities caused by propeller action are dependent on the propulsion system, installed engine power and duration of applied power (Prosser, 1986).

Field measurements of the water-level drawdown, and waves and currents produced by passing craft are the best means of determining bank protection criteria. In the absence of such data, values can be estimated using the procedures described by PIANC (1987). Craft under 40 tonnes navigating in small canals and rivers in the United Kingdom can produce waves of up to 0.4 metres high, but currents of up to 3 m s$^{-1}$ can be produced (CIWEM, 1989).

4.6.5 Evolution and characterization of river bends

Natural watercourses generally tend to form irregularly varying channels in their flood flow beds and flood plains. This phenomenon is explained by the fact that each river is a system striving for dynamic equilibrium, in which one of the components of change – in addition to the river slope – is the formation of river bends or meanders.

Many theories have been offered to explain the physical reasons for meandering. Although there are differences, most have the following points in common:
(a) One of the components of meandering is valley fill with sediment movement;
(b) Natural rivers strive to achieve or maintain a state of dynamic equilibrium;
(c) The nature of meandering, the development degree of bends and the frequency of their occurrence vary from river to river.

The primary task of river training is to find an optimal, self-stabilizing river course that is adapted to its particular nature. The artificial bends should be selected so that a new dynamic equilibrium can be established. To do so, it is indispensable to study the bends that are still in a natural state so as to become familiar with the river regime.

The sinuosity of river bends can be characterized in simple terms as a series of circular arcs (see Figure II.4.33). The following parameters must be determined:
$L$ – Arc length, as measured along the central line, between the two turning points;
$H$ – Bend length;
$A$ – Bend amplitude;
$R$ – Bend sinuosity or radius;
$\alpha$ – Central angle of the river bend.

Depending on the degree of its development, a river bend can be:
(a) A straight reach;
(b) A false bend, when the straight line connecting the two neighbouring turning points does not intersect the convex bank line, but remains between the two bank lines;
(c) A true bend, which in turn may be:
(i) An underdeveloped bend, if in each of the two neighbouring inflexion cross-sections, there is at least one point from which that of the other section is visible;
(ii) A developed bend, if $1.2 \ H < L < 1.4 \ H$ and $\alpha_i < 120^\circ$.

![Figure II.4.33. Definition sketch of river bend parameters](image)
The sinuosity characteristics of Figure II.4.33 can be plotted as a longitudinal profile or can be investigated as random variables by statistical methods.

The geometrical characteristics of the riverbed in each cross-section are the following:
(a) Area of the cross-section (F);
(b) Cross-section width (B);
(c) Wetted perimeter (P);
(d) Hydraulic radius \( R = \frac{F}{P} \);
(e) Average water depth \( H = \frac{F}{B} \).

The geometrical characteristics of the riverbed change both in time and along the river. On the basis of periodic riverbed surveys, the geometrical characteristics can be investigated either as functions of the water stage or, with relative frequencies, of the various variables computed for different river reaches. Figure II.4.34 is an example showing the width variation of the cross-section along the Danube river downstream of Budapest.

4.6.6 Determination of design discharges and stages

4.6.6.1 Determination of the design discharge for flood-bed regulation

Characteristic flood data can be determined and flood discharges, with various probabilities, can be computed by using the methods described in Chapter 5. The outputs of these computations are the basic data necessary for selecting the design discharge for flood-bed regulation.

In present practice, the design discharge is given as a magnitude of a given probability, or a given average return period, of the ice-free annual peak discharges. The probability depends on the demographic and economic conditions of the area to be protected.

4.6.6.2 Determination of the design discharge for mean-bed regulation

The dimensions of the mean bed are related closely to the flow and sediment regimes. Both regimes, and consequently, the evolution of the riverbed, are processes that are changing in time. The task is to determine the effective, or design discharge that has the greatest impacts on the natural and/or planned dimensions of the riverbed. Figure II.4.35 shows a groyne-fixed river flow, which should be the natural line of a meandering river.

Each of the geometrical parameters of the riverbed may vary in a different manner, depending on the duration of the various discharges. Thus, one discharge value will be the dominant one with respect to the width of the mean-flow bed, while another will be dominant for its depth. For each of the geometrical parameters, a discharge value can be found whose effect on that parameter will be the strongest, but there will be no single discharge that will equally form all riverbed variables and optimize them.
Since the sediment regime plays an important role in riverbed formation or design, the characteristics of sediment transport should be considered. See 4.8.

One method for determining the design discharge $Q_D$ at a given cross-section of a river may be applied graphically or numerically (see Figure II.4.36): the vertical axis of an orthogonal coordinate system indicates water stage $H(m)$ and the horizontal axis is calibrated for four different variables – water stage frequency $f (m^{-1})$, flow discharge $Q (m^3 s^{-1})$, average flow velocity $v (m s^{-1})$ and the product $P = \Delta f \cdot Q \cdot v (m^4 s^{-2})$, where $\Delta f$ is dimensionless (as $\Delta f = \Delta f(H) = [m][m^{-1}]$). In this coordinate system, the curves representing the relationships $Q(H)$, $v(H)$ and $f(H)$ are plotted first.

While $Q(H)$ and $v(H)$ are generally concave curves, as shown in Figure II.4.36, $f(H)$ is a more or less asymmetric histogram, or bell-shaped curve, whose basis is the vertical $H$ axis, and the area enclosed between the $f(H)$ curve and the $H$ axis is unity. The $H$ axis may then be subdivided into a sufficient number of equally spaced intervals of $\Delta H (m)$ within the area between the minimum and maximum water stages recorded. At the medium stage $H_i$ of each interval $\Delta H$, the values $Q_i = Q(H_i)$ ($m^3 s^{-1}$), $v_i = v(H_i)$ ($m s^{-1}$) and $f_i = f(H_i)$ ($m^{-1}$) are read from the respective curves and the products $\Delta f_i = \Delta H_i \cdot f_i$ are computed. Finally, for each water stage $H_D$, the product $P_D = Q_i \cdot v_i \cdot \Delta f_i (m^4 s^{-2})$ is calculated. This product is proportional to the kinetic energy of the flowing water, and the location of the resultant $P_D$ of the parallel (horizontal) powers, $P'$ is determined, for example, by using the graphical funicular polygon method or the numerical momentum equation, both of which are well-known methods in statics. At the water stage $H_D$, corresponding to the resulting power $P_D$, the required value of the design discharge $Q_D = Q(H_D)$ can be read from the curve $Q(H)$. The results thus obtained should be checked in river reaches that are presumed to be stable.

4.7 URBAN WATER RESOURCES MANAGEMENT
[HOMS I26, I81, K22, K70]

4.7.1 General

Urban water management is a broad term covering the management of water use, water conservation and impacts on the aquatic environment in urban areas. Urban development has an impact on water and the environment. Integrated urban water management is the development of water facilities by using approaches that combine urban planning and sustainable development. As part of urban planning, integrated urban water management is recognized as the most appropriate mechanism for providing infrastructure and services for water supply and the management of urban wastewaters, including storm water runoff.

4.7.1.1 Water sources and impacts

The design, maintenance and management of storm drainage systems is highly dependent on the origin of the water which, in an urban area, may be any of the following:
(a) Runoff from upstream areas;
(b) Runoff from adjacent areas;
(c) Baseflow from groundwater;
(d) Runoff from rainfall over the area considered;
(e) Tides and surges;
(f) Wastewater (sanitary, industrial and so forth).

Flooding caused by runoff from rural areas or from high groundwater levels is considered in other chapters. Chapter 4 focuses on the design and management of urban drainage systems to deal with surface runoff from local rainfall and its interaction with receiving water bodies.

Municipal and industrial water supply and management are related to urban drainage because they are the source of polluted domestic and industrial wastewater. Daily variations in the quantity and quality of wastewater from these sources need to be monitored because they serve as inputs to the following tasks:
(a) Drainage-system design, maintenance and rehabilitation;
(b) Design and management of wastewater treatment plants;
(c) Assessment of the impacts of polluted and treated water on receiving water bodies.

The monitoring and management of groundwater in urban water areas are very important because of the variety of ways in which human activities interact with the balance and quality of groundwater, which is often a major source of drinking water for urban areas. However, groundwater recharge in urban areas is generally reduced because of the increased percentages of impervious areas that cause lower infiltration rates and faster surface runoff. Furthermore, groundwater in urban areas is subject to pollution from both point and non-point sources.

4.7.1.2 Goals of integrated urban water management

The goals of integrated urban water management are as follows:
(a) Provide good-quality water in adequate quantities and meet domestic and commercial purposes under optimal economic conditions;
(b) Minimize pollution and other adverse effects on the environment, including adverse groundwater level changes;
(c) Minimize the costs of floods and damage caused by storms through adequate storm drainage based on the combination of improved drainage networks, real-time control of auxiliary structures (retention and detention basins, pumping stations and the like) and warning systems;
(d) Minimize the adverse effects of treated or untreated urban waters (domestic, industrial and storm) on receiving water bodies.

Managing urban drainage systems to meet these goals involves the following tasks:
(a) Evaluating the impact of urban development on the discharge and water quality of the basin under alternative scenarios and for different return periods;
(b) Designing and implementing control measures and storm-drainage practices to reduce the impacts;
(c) Implementing these measures through sound management.

4.7.2 Urban development impacts

Urban drainage catchments differ from rural catchments in many respects:
(a) Land-use patterns are different and generally better documented than in natural catchments;
(b) The percentage of impervious areas is higher;
(c) Unless special techniques for runoff reduction are applied, floods are generated rapidly with higher peaks;
(d) Water is drained from the catchments by a combination of surface collectors and underground drainage systems;
(e) Urban drainage basin areas are generally small, although in large metropolitan areas they can be large, featuring complex systems of buried pipes, pumping stations and, in recent years, large underground storage facilities.

Urban development changes land use (see Figure II.4.37), sharply increasing the percentage of impermeable area, such as roofs, streets and parking lots. It also introduces man-made drainage, such as conduits and channels, which modify the hydrological cycle by increasing overland flow and decreasing groundwater flow. Under this scenario, peak discharges increase (Figure II.4.37 (b)), as does the frequency of flooding. The higher flows from urban surfaces can carry with them greater loads of total solids, such as sediments and garbage, and pollution, which then degrade the water quality of the receiving waters.

Where an urban area is already developed, the solids produced in the basin come mainly from sediments and solid wastes washed from urban surfaces. In this case, total solids are a function of the frequency of solid waste collection and the cleaning of street
surfaces, as well as hydrological factors such as the frequency of rainfall events.

On rainy days, the surface wash load is derived mainly from litter and other surface contaminants. Table II.4.2 shows the variation of some water quality parameters for different land uses, as measured in cities in the United States.

Many diseases can be traced to poor water management. In the humid tropics, diseases and symptoms related to poor water supply, sanitation and drainage include diarrhoea, cholera, malaria, dengue and leptospirosis. The environmental conditions related to drainage which help to spread malaria are stagnant water, deforestation, soil erosion and flooding. Dengue is a disease found in warm climate which is spread by mosquitoes that live in clean, stagnant water that may be kept in or near homes (tyres, vases, and so forth) during the rainy season. Ponds or on-site detention systems should be carefully designed and monitored in such climates to avoid maintaining an environment favourable to this kind of disease.

4.7.3 Urban storm drainage design

The main design components of sewer systems are gutters, conduits, channels and detention or retention elements. The hydrological design of these parts is based on the calculation of the design maximum discharge or the hydrograph which integrates both the flood peak and volume. The methods used in design are generally based on assumptions regarding the rainfall-runoff relationship. There are two major methods:
(a) The rational method, which estimates only the peak discharge and assumes that the proportion of the rainfall that runs off is constant and that the rainfall intensity for any given duration is also a constant. These are reasonable assumptions for small basins of less than 2 km²;
(b) Flood hydrograph estimation, which computes the peak and volume of the flood event. This is likely to be important for reservoirs and takes into account large urban basins.

The main inputs of these methods in estimating the maximum discharge and its volume are design rainfall, land use within the upstream basin and other characteristics of that basin.

4.7.3.1 Design rainfall

Storms over urban areas are, as elsewhere, stochastic in nature. Therefore the design of the drainage system is based on storms of certain return periods. Rainfall depth for a certain return period is normally taken from rainfall intensity–duration–frequency curves that have been established for many cities (see 5.7). The choice of the return period for the design storm to be used as input for rainfall-runoff analysis depends on the importance of the area to be protected and the possible damage that might be caused by flooding.

Rainfall intensity varies greatly from a temperate climate to a humid tropical climate. Figure II.4.38 shows the one-hour duration rainfall of 11 gauges in a humid tropical region of Amazonia compared with the mean of the rainfall for gauges outside this region (subtropical and temperate). For the same return period, the difference in rainfall intensity is about 25 per cent, which might translate into a 50 or even 100 per cent increase in the design flood peak, depending on the design method used.

Table II.4.2. Median event mean concentration for Nationwide Urban Runoff Program, United States (Environmental Protection Agency, 1983)

<table>
<thead>
<tr>
<th>Constituent (mg/l)</th>
<th>Residential</th>
<th>Mixed</th>
<th>Commercial</th>
<th>Non-urban</th>
</tr>
</thead>
<tbody>
<tr>
<td>Biochemical oxygen demand (BOD)</td>
<td>10</td>
<td>7.8</td>
<td>9.3</td>
<td>–</td>
</tr>
<tr>
<td>Chemical oxygen demand (COD)</td>
<td>73</td>
<td>65</td>
<td>57</td>
<td>40</td>
</tr>
<tr>
<td>Total suspended solids (TSS)</td>
<td>101</td>
<td>67</td>
<td>69</td>
<td>70</td>
</tr>
<tr>
<td>Lead (Pb)</td>
<td>0.144</td>
<td>0.114</td>
<td>0.104</td>
<td>0.03</td>
</tr>
<tr>
<td>Total copper (cu)</td>
<td>0.033</td>
<td>0.027</td>
<td>0.029</td>
<td>–</td>
</tr>
<tr>
<td>Total zinc (Zn)</td>
<td>0.135</td>
<td>0.154</td>
<td>0.226</td>
<td>0.195</td>
</tr>
<tr>
<td>Total Kjeldahl nitrogen (TKN)</td>
<td>1.900</td>
<td>1.29</td>
<td>1.180</td>
<td>0.965</td>
</tr>
<tr>
<td>Nitrite (NO₂⁻) and nitrate (NO₃⁻)</td>
<td>0.736</td>
<td>0.558</td>
<td>0.572</td>
<td>0.543</td>
</tr>
<tr>
<td>Total phosphorus (Tp)</td>
<td>0.383</td>
<td>0.263</td>
<td>0.670</td>
<td>0.121</td>
</tr>
<tr>
<td>Soluble phosphorus (Sp)</td>
<td>0.143</td>
<td>0.056</td>
<td>0.080</td>
<td>0.026</td>
</tr>
</tbody>
</table>
Managing urban drainage in such humid environments may require the use of a more frequent return period for design, with a consequently higher risk of inundation.

4.7.3.2 Basin development

The size and level of development of an urban area must be considered when modelling an urban drainage system. In addition, the modelling will need to be undertaken on a range of scales. The system is generally a combination of minor and major drainage flow networks. The former serves the drainage of small areas (≤ 2 km²), such as site developments or condominiums, whereas major drainage is composed of large trunk drains and/or major urban streams. The upstream basins of these main streams may include both urban and non-urban areas.

In developed urban areas, the drainage system is well defined while, in undeveloped areas, natural drainage still works. When studying future scenarios for basins in undeveloped areas, it is necessary to derive the general outline of a future drainage system from the urban development plan.

In countries where cities are not growing because the population has stabilized, which is the case in some European cities, future urban scenarios are mainly related to the improvement of existing drainage and water quality. In most developing countries, however, urban development is dynamic and often uncontrolled. It is very difficult to manage the potential impacts of this development on the runoff in order to avoid environmental degradation and increased flood damage.

Any computation of discharge in urban basins must be based on existing major and minor drainage systems, plus an analysis of likely or planned future development scenarios. The computation of a design peak discharge for small drainage areas generally involves the rational method, despite its limitations with regard to the spatial and temporal variability of hydrological processes (Heaney and others, 2002). The approach to be used for the major components of the drainage system will depend on whether the basin is developed or undeveloped.

Undeveloped or developing areas: In a basin which is currently undeveloped or is subject to increasing urbanization, undeveloped areas will not have intricately planned streets and minor drainage systems, but will have an urban plan based on population density in the form of an urban master plan. Empirical relationships between population density and impervious areas can be developed to help set a design figure for the percentage of impervious areas (AI). Such a relationship has been derived for three major Brazilian cities, São Paulo, Curitiba and Porto Alegre (Campana and Tucci, 1994), which results in the following equation:

\[ AI = 0.489 \times DH \quad (4.14) \]

where \( DH \) is urban density expressed as inhabitants per hectare. This relationship is valid in the Brazilian context as long as \( DH < 130 \) inhabitants/hectare. Above this density, an impervious area of 65 per cent is assumed. It was developed for areas greater than 2 km² since for smaller areas there could be some distortion.

4.7.3.3 Design peak flow

The design peak flow can be estimated by using the following procedures:

(a) Flood frequency based on a flow series of adequate length;
(b) Empirical equations based on a regional flood frequency analysis;
(c) Design rainfall fed into a rainfall-runoff model in order to estimate the discharge.

In (a), stationary and representative peak flow samples are needed. Such data are not always available and there can be difficulties in achieving a stationary sample of flow events because of the continuing urbanization of the basin. As
regards (b), empirical equations must be developed for specific regions based on the regional data. It is generally recommended that these equations not be used outside the region in which they were developed. The rainfall-runoff procedure, (c), is the method most used in estimating the peak and hydrograph of rainfall with a selected return period. This approach offers simple methods which are used for small basins. These compute only the peak flow, as is the case with the rational method described below. Other methods attempt to estimate both the peak and the flow distribution in time, thus yielding a design hydrograph. These are also described more fully below.

4.7.3.3.1 The rational method

In small basins, the simple rainfall–peak flow relationship known as the rational method may be used. This uses the following simple equation for the peak flow:

$$Q = 0.278 \cdot C \cdot I \cdot A \quad (4.15)$$

where $Q$ is the discharge in $m^3 \cdot s^{-1}$, $C$ is the runoff coefficient; $I$ is the rainfall intensity in mm/h and $A$ is the basin area in km$^2$. The rainfall intensity is selected according to the return period $T$ (generally 2–10 years in minor drainage systems) and the rainfall duration $t$. $T$ is based on the design decision and the characteristics of the system modelled. In the rational method, $t$ is equal to the time of concentration of the basin.

Time of concentration ($t_c$) is the sum of the time it takes for the water to flow over the basin surface until it reaches the inlet ($t_b$) and the travel time through conduits and natural and constructed channels ($t_r$):

$$t_c = t_b + t_r \quad (4.16)$$

The value of $t_b$ can be estimated by empirical equations developed for surface flow. The flow is generally less than 60 m long; if longer, it tends to be concentrated in a swale, gutter or a small natural channel. It can be estimated by:

$$H (m) \quad \uparrow \quad (4.17)$$

where $t_b$ is in minutes, $C_S$ is the runoff coefficient used for a five-year return period, $L$ is the length of the overland flow in metres and $S$ is the average basin slope in per cent (SCS, 1975).

Runoff coefficient: This coefficient ($C$) is presented as the ratio of the total overland flow to the total design rainfall over the basin. It is a function of rainfall intensity, the spatial and temporal distribution of the rainfall, the extent of urbanization and soil characteristics, among other factors. The evaluation of a mean value of $C$ for a drainage area is a highly simplified but pragmatic representation of its water balance and the impact of urbanization.

In design, this coefficient is estimated using tables presented in the literature (ASCE, 1992) such as those reproduced in Tables II.4.3 and II.4.4. The coefficient can be modified for return periods greater than ten years multiplied by the coefficient $C_f$ presented in Table II.4.5.

The runoff coefficient of a basin can be estimated from the proportion of pervious and

<table>
<thead>
<tr>
<th>Table II.4.3. Normal range of runoff coefficients (ASCE, 1992)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Surface character</strong></td>
</tr>
<tr>
<td>Pavement</td>
</tr>
<tr>
<td>Asphalt and concrete</td>
</tr>
<tr>
<td>Brick</td>
</tr>
<tr>
<td>Roofs</td>
</tr>
<tr>
<td>Lawns, sandy soil</td>
</tr>
<tr>
<td>Flat (2%)</td>
</tr>
<tr>
<td>Average (2–7%)</td>
</tr>
<tr>
<td>Steep (&gt;7%)</td>
</tr>
<tr>
<td>Lawns, heavy soil</td>
</tr>
<tr>
<td>Flat (2%)</td>
</tr>
<tr>
<td>Average (2–7%)</td>
</tr>
<tr>
<td>Steep (&gt;7%)</td>
</tr>
</tbody>
</table>

Note: Ranges of $C$ values are typical for return periods of 2–10 years.
impervious areas. A weighted coefficient can be calculated by:

\[ C = C_p + (C_i - C_p)AI \]  

(4.19)

where \( C_p \) is the coefficient for pervious areas and \( C_i \) is the coefficient for impervious areas; \( AI = A_i/A_t \) represents the ratio of impervious areas to the total area.

The application of this type of equation to 44 small urban basins in the United States produced the following relationship (Schueler, 1987):

\[ C = 0.05 + 0.9 AI \]  

(4.20)

with a correlation coefficient of \( R^2 \) equal to 0.71. The hydrological data used were from two-year series and the values of the coefficients may be understood as relating to a two-year return period (Urbonas and Roesner, 1992). In this equation, if \( AI \) is considered to be 1.0, then \( C \) is 0.95, that is, an impervious coefficient of 0.95 with losses of 5 per cent. This could be caused by depression storage, evaporation from warm surfaces, antecedent moisture conditions, infiltration at surface junctions and data uncertainties.

In Brazil (Tucci, 2001), 11 basins have been used with areas ranging from 3.4 km\(^2\) to 106 km\(^2\) and 1 to 51 per cent of impervious area, resulting in:

\[ C = 0.047 + 0.9 AI \]  

(4.21)

with an \( R^2 \) of 0.92. The latter equation has coefficients very close to those of 4.20.

This coefficient changes in each flood, depending on the rainfall intensity characteristics and initial soil moisture conditions. In a rural basin, \( C_p \) may vary greatly; therefore, it is important to recognize that these equations were developed according to mean values and represent only the mean conditions from the recorded data.

Figure II.4.39 shows the runoff coefficients for many events for two scenarios in the Diluvio Basin: 1979–1982, with 19.7 per cent of impervious areas; and 1995–1997, with 40 per cent of impervious areas. This demonstrates that there is a relationship between the runoff coefficient and peak discharge: both rise with increasing rainfall.

### Table II.4.4. Typical composite runoff coefficient by land use (ASCE,1992)

<table>
<thead>
<tr>
<th>Description of the area</th>
<th>Runoff coefficient C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Business</td>
<td></td>
</tr>
<tr>
<td>– Downtown</td>
<td>0.70–0.95</td>
</tr>
<tr>
<td>– Neighbourhood</td>
<td>0.50–0.70</td>
</tr>
<tr>
<td>Residential</td>
<td></td>
</tr>
<tr>
<td>– Single family homes</td>
<td>0.30–0.50</td>
</tr>
<tr>
<td>– Multi-units, detached</td>
<td>0.40–0.60</td>
</tr>
<tr>
<td>– Multi-units attached</td>
<td>0.60–0.75</td>
</tr>
<tr>
<td>– Residential (suburban)</td>
<td>0.25–0.40</td>
</tr>
<tr>
<td>– Apartments</td>
<td>0.50–0.70</td>
</tr>
<tr>
<td>Industrial</td>
<td></td>
</tr>
<tr>
<td>– Light</td>
<td>0.50–0.80</td>
</tr>
<tr>
<td>– Heavy</td>
<td>0.60–0.90</td>
</tr>
<tr>
<td>– Parks, cemeteries</td>
<td>0.10–0.25</td>
</tr>
<tr>
<td>– Playgrounds</td>
<td>0.20–0.35</td>
</tr>
<tr>
<td>– Railroad yards</td>
<td>0.20–0.35</td>
</tr>
<tr>
<td>– Unimproved</td>
<td>0.10–0.30</td>
</tr>
</tbody>
</table>

Note: Ranges of \( C \) values are typical for return periods of 2–10 years.

### Table II.4.5. Flow coefficient correction factor (Wright-MacLaughin Engineers, 1969)

<table>
<thead>
<tr>
<th>Return period in years</th>
<th>Coefficient Cf</th>
</tr>
</thead>
<tbody>
<tr>
<td>2–10</td>
<td>1.00</td>
</tr>
<tr>
<td>25</td>
<td>1.10</td>
</tr>
<tr>
<td>50</td>
<td>1.20</td>
</tr>
<tr>
<td>100</td>
<td>1.25</td>
</tr>
</tbody>
</table>

Quantitative aspects of determining urban runoff are discussed in more detail in 5.10.

4.7.3.4 **Hydrograph methods: storm water simulation models**

These methods are based on rainfall-runoff models which calculate the flood hydrograph from the rainfall for a selected return period and certain time and space distribution. In addition to the rainfall, the initial state of the model variables, model parameters and other basin characteristics must also be known. The return period of the flood is generally assumed to be the same as that of the rainfall. However, approaches can be used to allow the values of other inputs, such as rainfall losses and temporal patterns, to vary stochastically.

These models generally contain two major modules: a hydrological module and a hydraulic module. The hydrological module is used to calculate the overland flow volume in time. The hydraulic module is used to calculate the transport of this volume through streets, conduits, channels and reservoirs in time and space.

The hydrological module employs the following tools:
(a) Coefficients, as in the rational method;
(b) Infiltration equations such as those of Horton and Green and Ampt;
(c) Empirical relationships, such as those developed by the United States Department of Agriculture Soil Conservation Service (SCS), now known as the Natural Resources Conservation Service. The first group of methods is biased when the model is applied to a magnitude of rainfall that is for larger or smaller than that used in its development. The second and third groups are more reliable and found in models such as that of the SCS (SCS, 1975) and HEC-1 (Feldman, 1995). The main simplifications of these models include a uniform space distribution of parameters and rainfall in each sub-basin.

The hydraulic module can be represented by the following types of equation:
(a) Storage and kinematic wave transport equations, which feature two major simplifications: they are used for free surface flow in pipes and channels, but do not take into account backwater effects, which are very common in urban environments;
(b) Diffusion and hydrodynamic equations for free surface flow: this type of model takes into account the backwater effects but cannot be used for flow under pressure, which occurs when the flow is greater than the design conditions;
(c) Hydrodynamic equations for systems with pipe flow under pressure and free surface otherwise. This model is mainly used for the simulation of flood scenarios or events above the design conditions.

These models aim to reflect differences in impervious areas across the sub-basins, overland flow characteristics, different times of concentration for sub-basins and flood-routing effects through the main channels and streams. Where urban development is dynamic over time, the model should be used to evaluate the impact of changes in urban density derived from future planning scenarios using relationships between impervious areas and urban density (Tucci, 2001).

Examples of models used for this purpose are Mouse (DHI, 1990), Hydroworks (HR Wallingford – Wallingford Software) and storm water management model, or SWWM (Huber, 1995).

Information on hydrological and urban characteristics is required to estimate model parameters and reduce planning and project uncertainties. During the 1970s and 1980s, there were significant advances in the methods or procedures for measuring rainfall and runoff (Maksimovic and Radojkovic, 1986), which have enabled the development and calibration of complex, often physically based, models for rainfall and runoff analysis and storm drainage system design (Yen, 1986). Although the drainage systems are generally designed to provide flood protection from storms of a specific probability, most of the present-day models can simulate the consequences of surcharged flow combined with surface flow on the streets (open-channel flow). More on such models can be found in 5.10.5.

Most urban studies in developing countries must be performed without the use of recorded hydrological data because the relevant data are either difficult to obtain or do not exist. Therefore, there is an urgent need for improved hydrological data collection in urban environments, especially in the humid tropics. Without these data, model parameters may present great uncertainties, which may result in higher urban drainage construction costs due to the oversizing of infrastructure, or costs associated with flooding caused by undersized drainage.

4.7.3.5 **Water quality**

Water quality models generally have a quantitative module simulating the discharges resulting from
rainfall, and a water quality module simulating the variation in water quality as expressed by parameters such as biochemical oxygen demand, nitrogen and phosphorus. The water quality module generally involves the following steps: pollution load evaluation, and transport, retention and control of the pollutant. Some of the models that have water quality components include SWWM, Mouse and Storm (HEC, 1977). The main difficulty with water quality simulation and evaluation is the lack of observed data for fitting model parameters. Consequently, validation is generally based on comparison with published information from elsewhere. Uncertainty analysis can be used to better understand the limits of the impacts and control measures needed to help make decisions for the management of urban developments.

4.7.4 Urban drainage control measures

The main goals of urban drainage are to decrease the frequency of flooding and improve water quality. Urban storm water management is mainly concerned with the distribution of the volume of water in time and space within the urban basin, taking account of urban development, hydraulic networks and environmental conditions (Urbonas and Stahre, 1993).

The key control measures are either structural or non-structural.

Structural measures: works designed to control the impact of floods on a major drainage system within a given urban development scenario. They are generally channel improvements and retention ponds.

Non-structural measures: land use and other regulations designed to limit the threat of flooding and flood warning, including the real-time forecasting of rainfall and of the likely impact of the forecast flood. Urban drainage regulations can be used to limit peak discharge downstream and reduce the degradation of water quality, taking into account social and economic conditions. Basic features of this type of regulation are to keep the peak discharge from the new development equal to or below the pre-development scenario and to set limits on impervious surfaces in each development. Public participation is essential for the development of effective regulations, which should include awareness-raising and educational programmes.

Source control measures for new developments have been included in the regulations of many countries (Urbonas and Stahre, 1993). Source control involves the provision of measures for storage near the location of the source of runoff, decreasing the need for conveyance increase downstream (Urbonas and Stahre, 1993). Some source control facilities are permeable pavements and parking areas, infiltration basins and trenches.

4.7.5 Urban drainage management

4.7.5.1 Principles

Experience gained from urban drainage planning in many countries has led to the establishment of some general urban drainage management principles (Urbonas and Stahre, 1993): (a) Management should be based on an urban drainage master plan for the municipality; (b) Public participation in urban drainage management should be increased; (c) Urban drainage control scenarios should take account of future city developments; (d) Urban drainage development should be based on cost recovery for investments; (e) An evaluation of flood-control measures should be undertaken for the whole basin, not only for specific flow sections; (f) Flood-control measures should not transfer the flood impact to downstream reaches but should give priority to source control measures; (g) More emphasis should be given to non-structural measures for flood-plain control, such as flood zoning, insurance and real-time flood forecasting; (h) Steps should be taken to reduce the impact of urban surface wash-off and other related urban drainage water quality problems.

In many developing countries, urban drainage practices do not fulfil these principles. The main causes are the following:

(a) Urban development in the cities occurs too fast and unpredictably. Generally this development starts downstream and moves upstream which increases the potential for negative impacts;
(b) Peri-urban areas are generally developed without taking into account the city’s regulations, or there are no city regulations;
(c) Peri-urban and risk areas – flood plains and hillside slopes – are occupied by low-income families and have no established infrastructure. Spontaneous housing development in risky areas in the humid tropics may be found in the following cities: on land prone to flooding – Bangkok, Bombay, Guayaquil, Lagos, Monrovia, Port Moresby and Recife; on hill slopes prone to landslides – Caracas, Guatemala City,
La Paz, Rio de Janeiro and Salvador da Bahia (WHO, 1988);
(d) Lack of appropriate garbage collection and disposal leads to pollution of the water and clogging of the drains. Some African countries have no urban drainage, and when systematic drainage does exist, it is often filled with garbage and sediments (Desbordes and Servat, 1988);
(e) Lack of institutional organization as a basis for developing urban drainage at the municipal level, hence no power of regulation, no capacity-building and weak administration.

4.7.5.2 Management practices

The main urban drainage policy requirements may be summarized as follows:
(a) Regulation should ensure that urbanization will not allow flood flows of a given return period to increase within the basin;
(b) Urban space should be reserved for detention (Figure II.4.40) or parks built within the river boundaries for storing flood volumes, sediment and trash detention and water quality improvement. If some of the impact of upstream urbanization cannot be controlled due to a lack of law enforcement, urban drainage policy may be used to limit to a minimum the transfer downstream of the impact. Instead of having the solid waste and sediments distributed in conduits or along the rivers and channels, they can be retained in specific places for cleaning, reducing maintenance costs. However, this is not always the best solution; therefore, each case should be evaluated on the basis of local conditions. Further guidelines for this form of integrated land and water management in urban areas have been prepared, for example, by Lawrence (2001).
(c) When the solution for flood control in the major drainage system is the use of storm sewers or increased channel capacity, the plan or design has to evaluate and limit the downstream impact of storm sewers or expanded channel capacity.

Based on these principles, urban drainage management should incorporate the following features:
(a) Prevention: planning urban space by taking into account urban drainage flood-plain areas in city development. Source control and non-structural measures are the main choices at this planning stage;
(b) Permanent institutional elements: regulation of minor drainage taking into account the increase in peak flow; regulation of land use in flood plains; tax incentives to protect conservation areas and existing drainage control areas; public procedures to check and enforce regulations based on local conditions; increased law enforcement at the site level when the area is already partially developed;
(c) Capacity-building: improve the technical capacity of local and state government personnel; create better working conditions so that skilled professionals can remain on the job; production of a city urban drainage manual; operation of a technical education programme for architects and engineers; general education of the population regarding relevant issues;
(d) Public participation: use public opinion polls as part of a campaign to involve the general public in the planning of urban drainage facilities, taking into account local requirements; consult the public through representatives of non-governmental organizations with regard to urban drainage plans and projects at all stages of development; increase public awareness of the impact of urbanization on urban drainage;
(e) More hydrological data: the lack of adequate hydrological and physiographic data is a chronic problem in the urban areas of developing countries resulting in the design of projects characterized by high cost or underperformance. A programme of data acquisition and development of methodologies for the use of data in the production of information for urban drainage is essential for sound urban drainage planning;
(f) Impact control: Structural measures for urban flood control may be developed sub-basin by sub-basin so as to decrease the impacts of urbanization with regard to water quantity

Figure II.4.40. Detention for urban drainage control, planning stage (Tucci, 2001)
and quality. In the planning process, rainfall-runoff and water-quality models can be used to assess the efficiency of the controls measures. The associated costs are generally distributed to the basin population based on the impervious area of their property.

4.7.6 Remote-sensing estimates for land use

Remote-sensing techniques play an important role in urban drainage design, particularly for the estimation of land use. This is discussed in Volume I, Chapters 2 and 4.

4.8 SEDIMENT TRANSPORT AND RIVER CHANNEL MORPHOLOGY

4.8.1 General

The transport of sediment by water flowing in rivers and channels is an important factor in the planning, design and operation of water management projects. It affects the life of storage reservoirs, the stability and conveyance of river channels, the design of structures that are in contact with the flowing water and the suitability of the water supplies for various uses. A proper assessment of the effects of sediment transport, and of the measures that may be necessary for its control, require knowledge of the processes of sediment erosion, transportation and deposition, and of their interaction with the hydrological processes in the catchments concerned.

This section is devoted to these erosion and sedimentation processes and their role in determining river channel morphology, while 4.10 addresses questions concerning hydroecology, in which channel morphology is a key factor.

4.8.2 Catchment erosion

Agents of erosion include wind, ice and gravity, but the most efficient one is running water. The processes by which water degrades the soil are complicated and depend on rainfall and soil properties, land slope, vegetation cover, agricultural practices and urbanization. The last two factors account for the most important effects of man’s activities on erosion.

Empirical equations have been developed for the determination of soil loss or sheet erosion from agricultural lands. One was developed by Musgrave for conditions prevailing in the United States (Chow, 1964). It was subsequently amended to apply to a wider range of conditions to yield the Universal Soil Loss Equation. This was then further developed to include erosion caused by construction and building. The result is the Revised Universal Soil Loss Equation:

$$ A = R \cdot K \cdot LS \cdot P \cdot C $$

(4.22)

where $A$ is soil loss in tonnes per hectare per year, $R$ is a rainfall erosivity factor; $K$ is a soil-erodibility factor; $LS$ a topographic factor composed of $L$, a factor dependent on the length of the slope, and $S$, the slope of the land surface; $P$ is a conservation-practices factor; and $C$ is a cover factor. Each factor is evaluated by using maps and tables derived from empirical data for the particular location and conditions.

Bare land and badlands may develop gullies with rates of advance that can be computed by empirical formulae containing parameters such as the drainage area of the gully, approach channel slope, rainfall depth and clay content of the eroding soil.

4.8.3 Channel erosion

Channel erosion is caused by the forces of the concentrated flow of water. Its rate depends on the hydraulic characteristics of channel flow and the inherent erodibility of channel materials. In non-cohesive materials, the resistance to erosion is affected by the size, shape and specific gravity of the particles and the slope of the bed. In cohesive materials, it also depends on the bonding agents. The relationships between the hydraulic variables and the parameters influencing the erodibility of channels are not fully understood and are often expressed by empirical formulae (Chow, 1964), (Maidment, 1992). Stream and river control works can accelerate channel erosion locally if they cause an increase in channel depth or flow velocity, change the direction of the flow, or reduce the natural sediment load. The latter effect occurs frequently below dams and may persist for many kilometres downstream. Procedures for measuring and computing bed material, suspended sediment discharge and sedimentation are discussed in Volume I, 5.5.

4.8.4 River systems

Rivers are formed along more or less defined channels, draining from the land the water that runs off from precipitation and from the melting of snow at high altitudes. They developed over the
ages. Along with water, they also convey sediment, washed down from the catchments and eroded from their own bed and banks. River systems and river processes are complex. For example, the inputs to a river reach are the water and sediment discharge, and the primary responses are the width, depth and velocity of flow, sediment discharge through the reach and the rate of sediment and water storage, which could be plus or minus, in the reach under consideration. Bed roughness and friction factors may be regarded as secondary factors; their values are interrelated with the depth and velocity of flow, sediment transport rate and to some extent, with the rate of scour and deposition.

Over geological time, a river evolves in such a way that it can in the long run transport the sediment delivered to it with available water runoff. Most natural channels are considered to be in regime flow when the major dimensions of their channels remain essentially constant over an extended period of time. The condition of regime flow does not preclude the shifting of channel alignment by erosion and rebuilding of the banks, but it requires balance between these factors. It requires that the sediment discharged from any given reach be equal to that which is introduced into the next reach. However, this does not mean that there is an invariant relationship between sediment discharge and water discharge. For most mobile bed streams, there will be a range of discharge values within which the stream can adjust with as much as a tenfold variation in sediment discharge by variation in bed forms – ripples and dunes. A concurrent variation is flow depth and velocity, without any appreciable changes in slope, channel width or average bed elevation. A stream may vary its channel dimensions locally, in time or space, without interfering with regime flow as long as these variations fluctuate around a balanced average. Indeed, this is how a river adjusts downstream of a confluence with a tributary which has different characteristics of sediment transport.

4.8.4.1 Aggrading and degrading streams

In certain sections of some streams where the amount of sediment introduced exceeds that which the stream can transport, the excess must be deposited. The stream bed is thus built up or aggraded. Conversely, if the rate at which sediment is introduced to a stream is less than its transport capacity, and its channel bed and banks are erodible, the stream will erode bed and banks to supply the deficiency. The major dimensions of aggrading or degrading channels remain in a constant state of change until equilibrium is established between sediment inflow and discharge.

4.8.4.2 River channel patterns

There are three main types of river channel patterns: straight, braided and meandering. These characteristics are considered from a plan view. Numerous factors influence whether a stream takes one form or other and their relationships are still not completely understood.

4.8.4.3 Straight channels

Straight channels are those that have a straight alignment. They generally occur when a channel slope is similar to a valley slope, or where steep slopes produce relatively high velocities. In the latter case, it is possible that the straight alignment results primarily from momentum that discourages turning.

4.8.4.4 Braided channels

A distinctive characteristic of braiding is multiple channels. There are, however, two types of multiple channel stream: one is the interlaced multi-channel stream separated by islands at low stages giving the appearance of braided hair. At high-flow stages, the islands may become submerged and the stream may flow from high bank to high bank (Figure II.4.41a). Another type of multiple channel stream is the distributary type found in deltas or debris cones. (Figure II.4.41b) These are generally

![Figure II.4.41. Braided channel (a) and distributary channel (b)](image-url)
aggrading channels that divide to follow separate courses, which finally disappear into sheet flow at the coast.

4.8.4.5 Meandering channels

Meandering channels follow a winding or tortuous course. They tend to shift continuously by local erosion and rebuilding of banks. Most problems arising in channel control concern meandering streams, as bank erosion occurs frequently with these. As solutions to these problems depend on knowledge of channel characteristics, numerous authors have studied and produced voluminous work on meandering rivers.

The basic stream meander is essentially a sinusoidal curve as shown in Figure II.4.42a. It is a dynamic form, tending constantly to shift its position by erosion of the concave bank and deposition along the convex bank of the bends. Under ideal conditions, a meander system will migrate downstream in an orderly progression along a central axis (Vanoni, 1975). The primary dimensions of a meandering system are its length, width and tortuosity ratio – also known as its sinuosity. Five primary factors determine these dimensions: valley slope, bank full discharge, bed load, transverse oscillations and degree of erodibility of the alluvium.

Ideal meandering systems seldom exist in nature. Individual meanders and overall systems of natural meandering streams tend to become distorted. A typical meandering stream Figure II.4.42b) is formed of numerous irregular bends of varying size and shape that resemble an ideal meander pattern only in respect to the alternating direction and continuing migration of the bends.

4.8.5 Flow regimes and bed forms

When the average shear on the bed of an alluvial channel exceeds the critical shear stress for the bed material, the material forming the bed starts to move, thereby disturbing the initial smooth bed. The nature of the bed and water surface change as the characteristics of flow and sediment change. The types of bed and water surface are classified according to their characteristics and are called regimes of flow (Garde and Ranga Raju, 2000). Bed form has an effect on flow resistance, sediment transport and turbulence.

4.8.5.1 Process of bed forms

Undulation and deformations of the mobile bed of channels are called bed forms. According to one school of thought, a small disturbance on an initially flat bed can, under certain conditions, affect the flow and local transport rate of sediment leading to the formation of troughs and crests. Bed deformation caused in that manner accentuates the disturbance, which in turn increases the rate of local scour in troughs and deposition over crests, leading to the formation of ripples and dunes. The growth of bed forms thus continues until a stage is reached when factors associated with the increased size of bed forms intervene and limit further growth.

Ripples and dunes thus achieve their optimum size. This is known as a lower regime flow and begins with the start of motion. The resistance to flow is large and sediment transport is small. The bed form is either ripples or dunes or some combination thereof. Resistance to flow is caused mainly by the form of the roughness. Plane bed, ripples and dunes are the bed forms in this range.

Under certain other conditions, the local sediment transport rate works in such a way that the size of troughs and crests is diminished, leading to the establishment of a flat bed. This is called upper regime flow. In upper regime flow, resistance to flow is relatively low and sediment transport is high. The usual bed forms are antidunes and a plane bed. Hence it is desirable to know beforehand what regime would prevail in a stream for a known flow condition.
The transition zone encompasses the bed forms that occur during the passage from lower regime to upper regime. This transition is not unique in loose boundary hydraulics. Figure II.4.43 shows bed forms arranged in increasing order of sediment transport rate. The processes concerned are described below (Simons and Richardson, 1961; Van-Rijn, 1984).

### 4.8.5.2 Plane bed

When average shear stress on the bed is less than the critical shear for material on the bed, no movement of material occurs on the bed. This regime is called plane bed without motion of sediment particles, and the laws of open channel flow on a rigid bed would be applicable in such a case.

### 4.8.5.3 Ripples

When the flow and hence shear on the bed are increased, the bed deforms in small three-dimensional undulations called ripples. They are triangular in shape with a flat upstream shape and steep downstream face. With ripples on the bed, material moves primarily along the bed as bed load. Water is clear and the water surface is choppy. Ripples move slowly in a downstream direction.

### 4.8.5.4 Dunes

As the discharge is further increased, ripples grow in size and are called dunes. Dunes are also triangular in shape but much larger than ripples. Flow separates behind each dune and produces a wake and high turbulence. As a result, there is greater energy loss and further material is thrown into suspension. Dunes also move in a downstream direction. The Froude number is much less than unity. The water surface appears to boil. This regime is known as ripples and dunes.

### 4.8.5.5 Transition regime

If the discharge is increased further, the dune length grows, height decreases and the dunes are partly washed out. The Froude number is 0.8 or close to unity. Water surface waves are in phase with the partly washed-out dunes. This is known as a transition regime and is generally very unstable.

### 4.8.5.6 Antidune regime

A further increase in velocity leads to the formation of antidunes in the channel, characterized by standing waves on the water surface and antidunes moving gradually upstream, even though sediment movement is downstream.

### 4.8.5.7 Implications of bed forms

Bed forms are important factors to be considered in river engineering for several reasons. Flow resistance can be mainly attributed to bed roughness, which in turn is dependent on bed form geometry. The movement of large sand dunes past structures such as intake wells and bridge piers can render them unsafe, owing to lower bed levels in the trough. Water intake and pumping sites may be affected when dune crests move past them. The intakes can then get buried or draw in excessive quantities of silt-laden water. In the measurement of river discharge, the passage of large sand dunes can affect depths, slopes and velocities at the gauging section and thus reduce the accuracy of measurements. When stage–discharge relationships are to be established, large moving sand dunes can cause scatter in plotted data because widely different gauge values can be obtained for the same discharge, depending on the position of the dune with respect to the gauging station. It is therefore important to know what bed form can occur at different flood stages in a river at a required location.
4.8.5.8 Boundary roughness caused by bed form

The effect of bed form is to increase boundary roughness. Variation in the roughness coefficient ($n$) in the Manning formula (see Table II.4.6) varies from 0.010 to 0.030 depending on flow regime, which means that lower regime flow ($\text{Fr} = 0.15$ to 0.37) would have an $n$ value in the range 0.01 to 0.013. Rivers change their bed form during floods and hence their roughness coefficients. It is reported that the Darcy–Weisbach friction factors for ripples and dunes bed are 4.5 to 8.7 times greater than those for flow over a flat bed with immobile sand grain roughness. If dunes change to transition or flat bed, however, the roughness coefficient can diminish.

Table II.4.6. Variation of Manning’s roughness coefficient $n$ with different bed forms (Simons and Richardson, 1961), expressed in m$^{-1/3}$ s

<table>
<thead>
<tr>
<th>Bed form</th>
<th>Regime</th>
<th>Approximate Froude number ($\text{Fr}$)</th>
<th>Approximate $n$ value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ripples</td>
<td>Lower</td>
<td>0.14–0.37</td>
<td>0.018–0.30</td>
</tr>
<tr>
<td>Dunes</td>
<td>Lower</td>
<td>0.28–0.65</td>
<td>0.020–0.040</td>
</tr>
<tr>
<td>Transitions</td>
<td>Transitions</td>
<td>0.55–0.92</td>
<td>0.014–0.030</td>
</tr>
<tr>
<td>Flat bed</td>
<td>Upper</td>
<td>0.70–0.92</td>
<td>0.010–0.030</td>
</tr>
<tr>
<td>Antidunes</td>
<td>Upper</td>
<td>&gt;1.0</td>
<td>0.010–0.030</td>
</tr>
</tbody>
</table>

The roughness coefficient $n$ is analogous to the friction factor $f$ in the Darcy–Weisbach formula:

$$V = \left( \frac{8g}{f} \right)^{0.5} (R_h S)^{0.5} \quad (4.23)$$

where $V$ is mean velocity, $g$ is acceleration due to gravity, $R_h$ is hydraulic mean radius and $S$ is channel slope.

A relation between friction $f$ and the roughness coefficient $n$ is given by:

$$n = \mu \cdot f^{1/2} \cdot R_h^{1/6} \quad (4.24)$$

where $R_h$ is hydraulic radius [m] and $\mu = \sqrt{\frac{1}{8g}} \approx 0.113$ (for the SI system).

This results from the identity:

$$\frac{V}{V_s} = \left( \frac{8}{f} \right)^{0.5} = \frac{R_h^{1/6}}{n \sqrt{g}} = \frac{C}{\sqrt{g}} \quad (4.25)$$

where $C$ is Chezy’s coefficient, $V$ is flow velocity, and $V_s = \sqrt{g \cdot R_h \cdot S}$ is the shear velocity.

4.8.5.9 Resistance relationship

Rivers are dynamic in nature and their behaviour is affected by natural events such as earthquakes, landslides or changes in the local climate. The importance of bed form in discharge measurements is well known as it affects alluvial roughness and thereby the resistance relationship. Methods to predict alluvial roughness are available in the literature (Yalin and Ferreria da Silva, 2001).

4.8.5.10 Prediction of bed forms

Criteria have been proposed by several investigators for ascertaining the kind of bed form such as ripples, dunes, transition, flat bed and antidunes. Most of these criteria are however based on flume data, and certain difficulties arise when they are applied to rivers.

Depths and velocities across rivers are rarely uniform and hence bed forms observed in different parts of the sections can also be different. Owing to the three-dimensional topography of bed forms, their characterization is possible only in a statistical sense and it may be necessary to use spectral density functions to obtain a meaningful representation of the geometry. Bed forms take some time to change as a result of changes in discharge and always lag behind by a certain time. In addition, the type of bed form is dependent, not only on depth, slope and bed material size, but also sediment supply. In a flume with a low sediment supply, the bed form increases in size from ripples to dunes with increase in velocity. However, in a river with increase in discharge, velocity increases along with the sediment concentration, resulting in a reduced bed form size and roughness coefficient. The variation in Manning co-efficient $n$ in a flume with increase in discharge is not comparable to that in river. Because of these difficulties, the current advice is to observe the bed form in a river at the required location by means of an echo sounder and to repeat these observations at different flood stages.

4.8.6 Transportation of sediments in channels

When the shear stress on the bed exceeds the critical shear stress for the given material, material on the channel bed starts moving. Depending on the gradation of channel bed material, sediment is transported near the bed by contact, saltation or in suspension. Sediment transport, sediment load and
sediment discharge are commonly used terms in river-engineering, which divides the sediment load into three: bed load, suspended load and total load. The term wash load is also used. This is related to catchments and is composed of particles whose size is finer than those found in the stream bed. For a detailed discussion on the following formulae and other sediment transport equations, please refer to the Manual on Sediment Management and Measurement (WMO-No. 948). Various measurement techniques are discussed in the Manual on Operational Methods for the Measurement of Sediment Transport (WMO-No. 686).

4.8.6.1 Suspended sediment transport

Fine, or suspended, sediments transported in rivers, originate mainly from the topsoil of the catchments and from the banks of the channels. However, fine sediments also originate from sewage and other return flows. Such sediments comprise about one third of the suspended-sediment load in the lower Rhine river, for example. A large portion of the transported material settles on to the flood plains (Guy, 1970), especially upstream of hydraulic structures. The settled material undergoes compaction and other physical and chemical changes that can sometimes prevent its re-erosion by flows that would have otherwise carried it. A decrease in the mean annual sediment transported per unit area of the catchment is generally found as the area of the catchment increases. The concentration of suspended sediment in runoff is described by formulae such as that of the National Research Council (1973), Negev (1972) and Beschta (1987):

\[ \log c_i = C \log Q + B \]  

(4.26)

in which \( c_i \) is the concentration expressed in weight per unit volume of water, \( Q \) is the water discharge, \( C \) is a dimensionless coefficient and \( B \) is a function of the rainfall depth, the antecedent discharge or some other meteorological or hydrological variable.

The concentration of suspended sediment varies within the channel cross-section. It is relatively high in the lower portion and may also be laterally non-uniform so that it will need to be sampled at various points or along several verticals of the cross-section to obtain its mean. The mean concentration should be evaluated to compute the total sediment weight-per-unit time when multiplied by the water discharge. The graph of suspended sediment against time generally has a peak that does not occur simultaneously with the peak discharge. This lag is a result of the specific conditions in a watershed and no generally applicable method has yet been found to evaluate this difference.

4.8.6.2 Bed-load transport

Coarse sediments, or bed load, move by sliding, rolling and bouncing along channels and are concentrated at or near the channel bed. The variables that govern transport are the size and shape of the particles and the hydraulic properties of the flow. As described in 4.8.5, the channel bed assumes different configurations exerting resistance to a wide-ranging flow of water and assumes a maximum value for the dune configuration. An empirical formula for the rate of coarse sediment transport proposed by Du Boys in 1879 (Chow, 1964) is still widely used today in many models. The formula is as follows:

\[ q_s = \frac{c}{\gamma} \left( \frac{\tau_0}{\gamma} - \frac{\tau}{\gamma} \right) \]  

(4.27)

where \( q_s \) is the sediment transport rate per unit width of the channel in kg s\(^{-1}\) m\(^{-1}\), \( \tau_0 \) is the shear stress at the channel bed in kg m\(^{-2}\), \( \gamma \) is an empirical value for the minimum \( \tau_s \) required for transporting the sediments considered, \( \gamma \) is the density of the water in kg m\(^{-3}\), \( c \) is a dimensional coefficient in kg m\(^{-3}\) s\(^{-1}\), \( S_e \) is the energy slope of the water and \( R_h \) is the hydraulic radius in metres which, for wide rivers, may be replaced by the mean depth of water. Values of the coefficients for equation 4.27 are given in Table II.4.7 (Chow, 1964).

Table II.4.7. Typical values of \( c \) and \( \tau_c \) parameters

<table>
<thead>
<tr>
<th>Classification</th>
<th>Mean diameter (mm)</th>
<th>( c ) (kg m(^{-3}) s(^{-1}))</th>
<th>( \tau_c ) (kg/m(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine sand</td>
<td>1/8</td>
<td>8 370 000</td>
<td>0.0792</td>
</tr>
<tr>
<td>Medium sand</td>
<td>1/4</td>
<td>4 990 000</td>
<td>0.0841</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>1/2</td>
<td>2 990 000</td>
<td>0.1051</td>
</tr>
<tr>
<td>Very coarse sand</td>
<td>1</td>
<td>1 780 000</td>
<td>0.1545</td>
</tr>
<tr>
<td>Gravel</td>
<td>2</td>
<td>1 059 000</td>
<td>0.251</td>
</tr>
<tr>
<td>Gravel</td>
<td>4</td>
<td>638 000</td>
<td>0.435</td>
</tr>
</tbody>
</table>

A more theoretically based formula was developed by Meyer-Peter in 1934 (Chow, 1964):

\[ q_s = \left( \frac{\gamma q}{B} \right)^{2/3} \cdot \frac{S_e - A \cdot d}{B} \]  

(4.28)

where \( q \) is the water discharge per unit width of the channel in m\(^2\) s\(^{-1}\), \( \gamma \) is the specific weight of water in kg m\(^{-3}\), \( S_e \) is the energy slope, \( d \) is the representative
grain size in metres, \( q_b \) is the bed-load discharge per unit width of the channel in kg m\(^{-1}\) s\(^{-1}\), \( B \) is a dimensionless constant that assumes the value of 0.40 in a consistent unit system and \( A \) is a dimensional constant that assumes the value of 17.0 in the SI system of units. If the transported sediments are of diverse sizes, \( d \) replaces \( d_{50} \), which is the mesh size through which 35 per cent of the weight of the bed load would pass. Equation 4.28 yields results that are reliable, particularly for sand-bed channels.

A second version of this formula has been developed to take into account the effects of dunes (Meyer-Peter and Müller, 1948):

\[
q_b = 8 \sqrt{\left( \beta \cdot \frac{\tau^*}{s - 0.047} \right)^3 \cdot (s - 1) g \cdot d^3} \quad (4.29)
\]

where \( q_b \) is the bed-load discharge per unit width of the channel in m\(^3\) s\(^{-1}\) m\(^{-1}\), expressed in grains volume (without the empty spaces), \( g \) is gravitational acceleration, \( d \) is the representative grain size and \( \tau^* \) is the dimensionless boundary shear stress, which is expressed by:

\[
\tau^* = \frac{R_h \cdot S}{(s - 1) \cdot d} \quad (4.30)
\]

where \( R_h \) is the hydraulic radius, \( S \) the slope of the channel and \( s \) is an adimensional factor, given by \( s = \gamma_e/\gamma_s \), \( \gamma_e \) is the specific weight of the sediment, and \( \gamma_s \) is the specific weight of the fluid. Finally, the coefficient \( \beta \) is a function of two Strickler’s numbers of the channel and of the grains (that is, after form drag caused by bed forms has been excluded) and is expressed by:

\[
\beta = \left( \frac{K_f}{K_{grain}} \right)^{5/2}.
\]

Strickler number \( K \) is the same as 1/n where \( n \) is the roughness coefficient in Table II.4.6.

4.8.6.3 Total load formulae

Total load carried by the stream is the sum of bed load, suspended load and wash load. However it is difficult to relate wash load to flow conditions. Wash load is generally absent in flume experiments; therefore, total load would be bed material load plus suspended load.

Relationships for estimating total load can be broken down into microscopic and macroscopic methods. In the former, bed load and suspended load are calculated separately, then added. Einstein’s method can be cited as an example (Vanoni, 1975). Macroscopic methods are based on the premise that, since the suspended load and bed load are essentially dependent on the same flow parameters, there is no need to estimate each separately. Instead, the total transport rate can be related to flowing fluid and sediment characteristics. The Engelund and Hansen (1967) method can be mentioned here, as it is simple:

\[
q_t = 0.05 \cdot \sqrt{(s - 1) \cdot g \cdot d^3 \cdot \left( \frac{K_f^2 \cdot R_h^{1/3}}{g} \right) \cdot (\tau^*)^{5/2}} \quad (4.31)
\]

where \( q_t \) is the rate of sediment transport in m\(^3\) s\(^{-1}\) m\(^{-1}\), while \( s \), \( d \), \( g \), \( K_f \) Rh and \( \tau^* \) are the same as in equation 4.30.

Another formula for total load is that of Van Rijn (1984). The advantage of the Van Rijn method is that it allows a separate calculation of the bed load transport and the suspended sediment transport. While suspended sediment is a complex topic, and therefore not included in this publication, the bed load transport formula is provided as follows:

\[
q_b = 0.053 \cdot \frac{T^{2.1}}{D_50^3} \cdot \sqrt{(s - 1) \cdot g \cdot d_5^3} \quad (4.32)
\]

where \( q_b \) is the bed load transport rate per unit width, \( s \) is the same as in equation 4.30, \( g \) is the gravitational acceleration, \( d_{50} \) is the representative bed material size, \( T \) is the transport stage parameter:

\[
T = \frac{(U^2 - U_{cr}^2)}{U_{cr}^2} \quad (4.33)
\]

\[
D_s = d_{50} [(s-1) \cdot g / \nu^2]^{1/3}, \nu \text{ is the kinematic viscosity and } U_c \text{ is the bed shear velocity given by:}
\]

\[
U_c = \frac{C_f}{C_{grain}} \sqrt{g \cdot R_h / S} \quad (4.34)
\]

where \( S \) is the slope, \( R_h \) is the hydraulic radius, \( C_f \) and \( C_{grain} \) are the Chezy coefficients of the channel and grains, respectively, and \( U_{cr} \) is the critical bed shear velocity, given by the Shields diagram (see Manual on Sediment Management and Measurement (WMO-No. 948), 3.2).

4.8.6.4 Sediment transport on steep slopes and debris flow

In a steep catchment, different sediment transport processes may occur. During a flood event, discharge can increase to such a level as to destroy the armour layer of stream or torrent bed. As a result, fluvial transport of the bed material will start. In addition, sediment may be supplied to the channel from slope failures; thus sediment availability could be sufficient for the flow to move sediment in rates close to its transport capacity. At very high
sediment concentration, sediment moves in sloughs and flow becomes unsteady. At the front of the wave flow, the particles are more or less uniformly distributed over the flow depth, while the mixture behind the front may become more diluted. At the end the coarse particles are concentrated near the bed (Rickenmann, 1991).

Erosion can increase as a result of changes brought about by earthquakes or lava eruptions. Ash, clay, coarse gravel, boulders, trees, loose rocks and anthropogenic material are transported by flowing water in terrain such as debris or mud flows. Debris flows, like flash floods, are fast moving and occur in a wide range of environments. A debris flow has the consistency of wet concrete and moves at high speeds of 15 m s\(^{-1}\) or even faster. Debris flow commonly occurs in gently sloping alluvial fans, cone- to fan-shaped land forms created over thousands to millions of years by the deposition of eroded sediment at the base of mountains.

The measurement of sediment transport and debris flow in such cases is very difficult, but research studies are being carried out in France, China, Japan, the United States and the Russian Federation, where large areas are affected by intense erosion. Gravel bed transport, debris flow and mud flow are topics of recent interest and the work of Thorne and others (1987), Coussot (1997), Zhaohui Wan and Zhaoyin Wang (1994) provide further information.

4.8.6.5 Sediment transport in gravel-bed rivers

In mountain rivers, bed-load discharge accounts for a relatively large proportion of the total discharge. Estimates of sediment transport in gravel-bed rivers are limited due to problems associated with sampling of bed load and bed material in the field, extreme non-homogeneity of the bed material and non-equilibrium bed-load transport. On the basis of flume data using sediment sizes up to 29 mm and slope up to 20 per cent, an equation has been derived (Smart, 1984) for bed-load transport as follows:

\[
\frac{q_{BV}}{g(s - 1)d_a^3} = 4 \left( \frac{d_{90}}{d_{90}} \right)^{0.2} \cdot S^{0.6} \cdot \left( \frac{V}{V_s} \right) \cdot \tau_s^{0.5} \left( \tau_s - \tau^*_c \right)
\]

(4.35)

where \(q_{BV}\) is volumetric bed-load transport per unit width; \(S\) is the channel slope; \(g\) is gravitational acceleration; \(d_a\) is the arithmetic mean size; \(d_{90}\) and \(d_{30}\) indicate bed material size finer than 90 and 30 per cent, respectively; \(V\) is average velocity, \(V_s\) is shear velocity of flow, \(\tau^*\) is adimensional shear stress (as in equation 4.29) and \(\tau^*_c\), dimensionless critical shear stress, corrected to take into account the slope, expressed by:

\[
\tau^*_c = \tau^*_{0c} \cdot \cos \alpha \left( 1 - \frac{\tan \alpha}{\tan \phi} \right)
\]

(4.36)

where \(\tau^*_0\) is the critical Shields parameter, \(\alpha\) is the angle of the slope so that \(S = \tan \alpha\), and \(\phi\) is the angle of repose of a submerged bed material. Sediment transport in rivers is a subject that has interested many researchers for which other texts (Raudkivi, 1998; Yalin, 1992) could also be used for reference. These formulae are based on empirical, semi-theoretical and theoretical equations, checked with laboratory data. Owing to the non-availability of reliable data from natural streams, however, little field data was used. Results of these formulae often differ enormously. It is difficult to ascertain which formula yields the most realistic results. Selection of a particular formula or set of formulae requires calibration with observed data for a particular river system.

4.8.7 Sedimentation

Suspended sediments are deposited according to their settling velocity. A relationship between the grain size and the settling velocity is shown in Figure II.4.44. Coarse sediments deposit first, then interfere with the channel conveyance and may cause additional river meanders and distributaries. Sediment entering reservoirs deposits and may form deltas in the upstream part of reservoirs. The deposited sediment may later be moved to deeper parts of the reservoir by hydraulic processes within the water body. As the area of flowing water expands, the depth and velocity decrease; eventually even fine sediments begin to deposit. A significant concentration of suspended sediment may remain in the water column for several days after its arrival. This may interfere with the use of the stored water for water supply, recreation and other purposes.

Not all of the sediment will be deposited in reservoirs, however. A large portion of it may remain in the upper zones of the catchment, some may be deposited upstream of the reservoirs and the released water carries some downstream of the dam. The sediment-trapping efficiency of a reservoir depends on its own hydraulic properties, those of the outlet and the nature of the sediment. The density of newly deposited sediments is relatively low but increases with time. The organic....
component in sediment may undergo changes that may reduce sediment volume and enhance biochemical processes in the stored water. For further information, see 4.9 and 4.10.

4.8.8 Sediment control measures

Sediment control measures fall into two broad categories: land-treatment measures for watershed protection and structural measures. Detailed descriptions are provided by Vanoni (1975). The aim of land-treatment measures is to reduce erosion in the watershed, and thus the rate of sediment formation, by improving the protective cover on the soil surface, diminishing surface runoff and increasing infiltration rates. These measures include the following:

(a) Land management based on agronomy and forestry, such as the use of crop rotation and the exclusion of grazing on critical runoff and sediment-producing areas;

(b) Appropriate field practices such as contour farming on sloping land, the development of gradient terraces on steep slopes and the grading and lining of natural waterways, irrigation and drainage ditches, and depressions.

Structural measures are aimed at providing protection beyond that afforded by land-management measures. They include channel improvement and stabilization works, reservoirs, debris and sediment basins, levees, dykes, floodways and floodwater diversions.

4.9 WATER QUALITY AND THE CONSERVATION OF AQUATIC ECOSYSTEMS [HOMS K55]

4.9.1 General

Water resources projects should be designed and operated in an environmentally friendly way. In addition, they should comply with water-quality standards, thus avoiding detrimental effects on aquatic ecosystems and on water quality. This is the subject of this subsection of the Guide. The next subsection, 4.10, addresses the broader question of the environmental management of rivers in the context of river morphology and ecology, focusing on the main impacts that water resources projects can have on river ecosystems and the methods commonly applied to reverse or mitigate them.

There are close relationships between some quantitative characteristics of water bodies, such as the flow regime and dilution capacity in rivers, or the flushing time and stratification patterns in lakes, and their ecological functioning and water quality. As water resources projects generally alter some of these quantitative characteristics, it should be possible to estimate or predict the environmental impacts when these relationships are well understood and defined. Unfortunately, such relationships can be very complex, and in some cases are only known in qualitative terms. Moreover, the data required to parameterize them are rarely available in practice. Therefore, it is natural that only rough estimates can be made of water quality and a project’s environmental impacts.

Some measures to protect water quality and aquatic ecosystems were recommended by the United Nations at the International Conference on Water and the Environment: Development Issues for the Twenty-first Century (United Nations, 1992). Recommendations for the environmentally friendly design and operation of water resources projects, as well as mitigation, rehabilitation, and restoration of existing projects and further detailed references can be found in Petts (1984), Gore and Petts (1989), and the World Commission on Dams (2000). Recommendations relating to hydropower and irrigation dams are available in Brookes (1988) and Gardiner (1991), those regarding channelization...

4.9.2 Relationships between water quantity and water quality

4.9.2.1 Streams and rivers

A significant proportion of the variability in river water quality can be related to variations in river flow. The effects of changes in river discharge on the concentration and load of substances are numerous, and may counteract each other. An increase of river flow generally leads to the following developments:

(a) Enhanced dilution of pollutants entering with wastewaters;
(b) An increase in suspended solids derived from surface runoff and disturbance of bottom sediments;
(c) The release of materials adsorbed by, or precipitated in, sediments such as phosphates and heavy metals;
(d) Higher demand for biochemical oxygen caused by stirring up reducing substances from the riverbed;
(e) Decreased ratio of groundwater to surface runoff in the river flow, generally resulting in a lower pH;
(f) The washing out, and subsequent reduction of benthic organisms and in residence times;
(g) Attenuated effects of sudden inputs of pollutants;
(h) The reduced absorption of solar radiation and a related decline in water temperature and photosynthetic activity;
(i) Greater turbulence and better aeration leading to higher levels of dissolved oxygen in conjunction with lower temperatures.

The sequence and time of occurrence of high flows are critical in determining the extent of many of these effects. A second flood wave, following shortly after a first, may contribute little to the effects of the first flood. Thaw and rain after a long period of frost may lead to a sudden influx of road de-icing salts and may cause significant sodium and chloride peaks, despite the rise in flow (see earlier reference to these factors in 4.7). The land use, soil type, land cover and other characteristics of the portion of the basin in which the flood-generating runoff originates are other factors affecting the magnitude of water quality changes caused by high flows.

When the rise in river flow results in significant flood-plain inundation, a number of additional water quality effects may follow. Most significant among them are the following:

(a) Flood attenuation related to additional valley and bank storage, leading to a reduction in downstream flood flow, hence lowering the various effects listed previously under (a) to (i);
(b) Increase in the water surface-to-volume ratio, resulting in expanded opportunities for solar-radiation absorption and increases in water temperature and photosynthetic activity;
(c) Reduced flow velocity in the flood plain, leading to decreased re-aeration and deposition of potentially contaminated suspended solids outside the main river channel;
(d) Intensive contact with previously deposited sediment, various types of soil structures, dumps, wastewater treatment plants, industrial chemicals and so on, that can lead to river pollution.

In general, low-water periods produce opposite effects to those caused by flow increases. Further, low-flow periods are often accompanied by a relatively high diurnal variation in water quality characteristics, for example, dissolved oxygen, carbon dioxide, pH and temperature. In arid climates, the effect of evaporation on the concentration of various substances in the water can be significant. In cold climates, low-water periods in winter may also be periods of oxygen deficit whenever the ice cover interferes with the re-aeration process.

4.9.2.2 Large lakes and reservoirs

Thermal stratification is a result of natural factors. However, thermal pollution and increased water temperatures caused by flow reduction can be a causal or contributing factor (see 4.9.5.4). Figure II.4.45 shows a representative profile of the summer stratification in a large storage reservoir. Thermal stratification can lead to dissolved oxygen stratification, particularly in nutrient-rich meso and eutrophic lakes and reservoirs, as well as to the stratification of other dissolved substances. In the epilimnion or upper layer of water, the water is warmer in summer and its quality is generally better. In the upper layer, one may expect reduced silicate content following increases in diatom abundance, decreased hardness from direct inputs of precipitation water and, most importantly, increased dissolved oxygen caused by atmospheric exchange.
and photosynthesis by phytoplankton and macrophytes.

In the hypolimnion, or lower layer of water, the water is colder in summer and has a reduced concentration of dissolved oxygen. Various potentially harmful substances frequently accumulate in this layer owing to deposition on the bottom, adsorption on sediment and ingestion by living organisms which, when they die, decompose on the bottom of the lake (see 4.9.5.3). Anaerobic decomposition of algae and other organisms may occur in the hypolimnion. One may expect the hypolimnion to show trends of an increasing concentration of ammonia and hydrogen sulphide, a reduction in nitrate and sulphate concentration, an accumulation of sediment and occasionally of heavy metals, and a periodic increase in iron, manganese and phosphate concentrations.

During the turnover caused by the seasonal cooling of the surface layer of the lake, a convective circulation takes place, resulting in vertical mixing of the lake and a uniform temperature. In deep lakes and reservoirs with a large hypolimnion volume, these turnover events can lead to fish-kills and other problems, because a large volume of low-quality water is mixed with the higher-quality epilimnetic water.

In addition to the aforementioned effects, the following developments can be expected:
(a) In large lakes and reservoirs, organic matter is biodegraded to a large extent because of long residence times;
(b) Variations in lake water quality are dampened out for the same reason;
(c) Water quality in the rivers flowing out of a reservoir depends largely on the occurrence of stratification and the depth at which the intake structure is located, since rivers flowing out of natural, unregulated lakes draw epilimnetic water.

### 4.9.3 Effects of water resources projects on water quality in streams and rivers

#### 4.9.3.1 Dams and weirs

Dams – and to a lesser extent, weirs – generally have the following effects on water quality in the upstream reach of a river by raising upstream water levels:
(a) Intensification of self-purification processes because of increased residence time in the reach and more deposition of suspended solids, which results in increased solar-radiation absorption and changes in the sediment characteristics of the riverbed;
(b) A rise in water temperature and phytoplankton production, greater oxygen consumption and increased day-night fluctuations in oxygen, pH and carbon dioxide as a result of (a).

Fish migration may be disturbed both by the physical barrier and changes in water quality. Changes in stream bank or shoreline vegetation, which are governed by local topography, climate and water-level variation, may also affect water quality. For example, water turbidity may be increased in reservoirs with fluctuating levels. In cold climates, dams and weirs create favourable conditions for an extended duration of ice cover in upstream reaches. This leads to decreased re-aeration. Further effects where large storage volumes are concerned may result from thermal stratification. Increased pollution retained in the reservoir may lead to eutrophication and anaerobic conditions (see 4.9.5.1 and 4.9.5.2, respectively).

The effects of a dam or weir on water quality in the downstream river reach depend on the water residence time in the impoundment, where this is calculated as the ratio of storage volume to streamflow. They also depend on the stratification and dam design and operation, particularly the depth at which intake structures are located in relation to the hypolimnion. The most important effects of a dam or weir are as follows:
(a) Reductions in suspended solid load, pollution load and turbidity;
(b) Changes in the chemical characteristics of the water – often a lower concentration of dissolved oxygen and nitrates – and increases in phosphate, carbon dioxide and hydrogen sulphide.

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**Figure II.4.45. Representative profile showing summer stratification in a large storage reservoir with a high dam**
the latter particularly when anaerobic conditions prevail upstream;
(c) Lower summer water temperatures and higher winter water temperatures, with major effects on invertebrate and fish communities downstream;
(d) Less day-night temperature fluctuations to which the river flora and fauna must adapt.

4.9.3.2 River training works

River training generally involves a deepening and straightening of the river channel for various purposes, including navigation, flood control, land-use improvement and erosion protection. This results in changes in the geometric and hydraulic characteristics of the river channel, and in some cases, of the flood plain as well. For further information, see 4.6.

When river training is done for navigational purposes, it generally involves the construction of navigation weirs and locks. In addition to the effects of weirs (4.9.3.1), the training works and the operation of navigation canals lead to increased turbidity and mixing of the water and aeration from the mechanical effects of the moving boats. However, the boats are a source of routine and accidental pollution, and can re-suspend contaminated sediments from the bottom. The dredging of navigation lanes can also cause similar problems.

In other cases, river-training works lead to a reduction in self-purification processes because the straightening of the banks eliminates stagnant water zones both as areas of self-purification and as a favourable environment for animal and plant life. The reduced surface-to-volume ratio leads to a reduction in the solar-radiation absorption and re-aeration. The loss in re-aeration may be partly compensated when river training produces higher water velocities.

4.9.3.3 Flow reduction and augmentation

In addition to the flow-regulation effects of dams, many water resources projects involve downstream flow reductions resulting from diversions for various water supply purposes or augmentation from inputs of water coming from sources outside the basin.

When the extracted water undergoes treatment and the resulting sludge and residues are returned to the donor river, or when water is diverted from the less polluted portions of the river cross-section, the diversion effects are equivalent to a reduction in flow or to a pollution input (see 4.9.5). The disposal of sludge and residue is generally the focus of regulations and legislation relating to the quality of effluents. These differ widely from country to country.

The effects of flow augmentation depend mainly on the quality of the additional water as compared to that of the river water. An addition of water of poorer quality is equivalent to a pollution input, as is the net effect of a project diverting water from a cleaner tributary.

4.9.4 Effects of water resources projects on water quality in large lakes and reservoirs

Water quality in large lakes and reservoirs may be improved or degraded by water resources projects. Where such projects involve a withdrawal of water of a better-than-average quality, for example from the epilimnion, they will generally worsen the water quality in the lake. The same is true when water of poorer-than-usual quality is pumped into the lake or reservoir. As explained in 4.9.2, the quality of the water flowing out of a reservoir depends on whether or not stratification occurs, and on the depth at which the intakes are located.

Water quality in a large reservoir depends to a large extent on the characteristics of the underlying terrain before flooding and on the treatment applied to it. If the future reservoir bottom is covered by soil with rich organic content or humus, the latter is leached after the reservoir is filled and accelerated eutrophication may result (see 4.9.5.1). This may be avoided by the removal of vegetation and soil prior to flooding, although this is a costly operation.

4.9.5 Water quality changes caused by pollution

4.9.5.1 Eutrophication

One of the most common forms of pollution is excessive concentrations of nitrogen and phosphorus nutrients originating in urban wastewaters or rural runoff. This generally results in the rampant growth of algae, particularly in areas with low water velocity. The subsequent decrease in dissolved oxygen concentrations can lead to significant reductions or even the disappearance of a number of plant and animal species. This is known as eutrophication. It is a natural process that marks the maturing and ageing of lakes. However, under conditions not involving man's activity, this may
take hundreds or thousands of years, depending on lake size, hydrological conditions and land cover in the basin. Civilization is responsible for accelerated cultural eutrophication in a large number of lakes all over the world.

Eutrophication and its causative factors are a major water quality problem. Considerable research devoted to the study of cultural eutrophication has resulted in the availability of many quantitative criteria and models for assessing its development. Further, different methods have been developed to help improve the condition of culturally eutrophied lakes. Although the lack of many elements can limit primary productivity, nitrogen and phosphorus are the ones most likely to limit the growth of algae in natural waters. In some countries, attempts to stop the advance of eutrophication have been made by banning the use of phosphorus compounds in detergents and introducing advanced, tertiary treatment processes for the removal of phosphorus and nitrogen.

The effects of eutrophication are reflected in striking changes in the affected lake ecosystems. Highly polluted environments have few species. When pollution is caused by toxic substances, the number of individuals surviving in each species is low, sometimes extremely low. However, when there is an excess of nutrients, a handful of species can reach large population numbers, owing to their increased productivity, but this is always matched by a decrease in diversity, because of the extermination of many other species that cannot withstand deteriorating environmental conditions.

Further details on cultural eutrophication and possible solutions can be found in Henderson-Sellers and Markland (1987), Harper (1991) and Ryding and Rast (1989).

4.9.5.2 Organic matter and self-purification

A large proportion of polluting substances of municipal, industrial and, particularly, agricultural origin consists of organic matter. A number of phenomena occurring in natural waters tends to transform this organic matter into more or less innocuous inorganic nutrients, a process known as self-purification. Some of these nutrients are recycled by algae and other producers, which generate secondary organic pollution upon dying, such as in eutrophication. Before the biological degradation leading to self-purification can take place, the organic substances dissolved in the water must be adsorbed and concentrated on the surface of solid particles. Adsorption can take place on the solid particles on the river bottom, banks and macrophytes, and on suspended solids.

Most biological degradation is associated with oxygen consumption, which is the key factor in the self-purification process. When oxygen consumption in water proceeds so rapidly that it exceeds the rate by which oxygen is replenished from the air or by oxygen-producing biological activities, namely photosynthesis, the aerobic self-purification capacity of the water body is exceeded. This occurs when one or more of the following conditions occur:

(a) The load of organic matter exceeds the self-purification capacity;
(b) Biological degradation processes are accelerated by certain factors, for example temperature rise;
(c) Oxygen replenishment is diminished by thermal stratification, ice cover or other causes.

When the self-purification capacity is exceeded, water anoxia occurs, and the decomposition of the organic matter generally continues under anaerobic conditions. This kills most metacellular organisms and interferes with many uses of the water body. The use of water for recreation and fisheries is impossible under such conditions and it may be much less desirable for other uses, such as water supply.

4.9.5.3 Adsorption and accumulation of pollutants

Some harmful substances are adsorbed on organic and inorganic suspended solids. When the latter settle on the bottom, these toxic substances are temporarily removed from the main body of water. Organisms are also capable of concentrating a number of organic and inorganic pollutants through biochemical processes. For example, the concentration of some pesticides in aquatic organisms can reach levels up to 300 000 times higher than those found in the corresponding water environment.

However, owing to physical and biological processes, substances absorbed and accumulated by organisms may be returned subsequently into the water body in solution or in particulate form. The concentration of pollutants by different levels of the organisms is of particular significance because they are in the food chain and bioconcentrated pollutants are passed from one level of organism to another in increasingly higher concentrations. Such a process, called biomagnification, is responsible for mercury poisoning related to the well-known Minamata disease.
4.9.5.4 Thermal pollution

Thermal pollution is defined as an increase of the temperature of a water body over the natural level caused by the release of industrial or municipal wastewater – in particular, cooling water from nuclear and thermal power plants and other industrial processes.

The effects of thermal pollution on water quality are complex and relate to the effects of higher temperatures on the viscosity of water, its decreased solubility for oxygen and increased chemical and biological activity. Thermal pollution may also be a contributing factor in thermal stratification. As a result of thermal pollution, the period of biological productivity is lengthened, which leads to an increased load of organic pollution. In addition, certain species of green algae are replaced by blue-green algae, which transmit to the water undesirable characteristics of smell, taste and toxicity.

As mentioned previously, self-purification processes are accelerated by higher temperatures, and thus by thermal pollution, to the extent that acute oxygen deficits may occasionally occur. In winter, ice formation is delayed by thermal pollution and this broadens the possibility of re-aeration. Because aquatic animals are ectotherms, that is, cold-blooded, water temperature is a vital influence in their growth, reproduction, and survival. Most aquatic invertebrates and fish are adapted to narrow temperature regimes; any departures from the natural state caused by thermal pollution or reservoir releases of cold hypolimnetic waters can exterminate species from a river reach.

4.9.6 Measures to reduce effects of pollution on water quality

Such measures can generally be divided into two groups: preventive and corrective. Whenever feasible, preventive measures should be applied because they are more economical.

4.9.6.1 Preventive measures

Preventive measures consist primarily in removing pollutants at the source. Treating wastewaters, changing industrial processes, altering the chemical composition of certain industrial products by eliminating phosphorus compounds from detergents and artificially cooling industrial wastewaters are means of doing so. If pollution originates from diffuse sources such as pesticides, herbicides, fertilizers and uncontrolled urban waste, and is washed into a river or lake from the land surface, pollution abatement can be achieved only by changing the practices that lead to the uncontrolled spreading of pollutants and adopting measures to reduce runoff and soil erosion.

Significant pollution is generated through soil erosion. Its prevention requires adequate forestry management and construction and farming practices. Finally, pollution from leachates, originating from garbage dumps, may be significant at the local level. This can be avoided by ensuring that such dumps are appropriately located and designed.

4.9.6.2 Corrective measures

Reducing pollution in water bodies after pollutants have reached them is often difficult and costly. In most cases, it is only possible to treat the water diverted from the water body for specific purposes, such as for domestic or industrial water supply. However, in special circumstances, remedial work can be carried out for the whole water body. Where rivers are concerned, remedial measures consist mainly of artificial re-aeration or oxygenation, or the dredging of settled pollutants. Measures targeting lakes and reservoirs include the following:

(a) Emptying the lake regularly between late autumn and early spring so as to expose organic matter directly to the air and permit aerobic decomposition of the organic matter. This is more feasible for reservoirs and small ponds than for large natural lakes;
(b) Dredging the bottom of the lake mechanically or by suction in the areas that contain the highest concentrations of organic and pollutants. Disposal of this material can be a challenge, however;
(c) Forced re-aeration by compressed air in the de-oxygenated layers;
(d) Harvesting and disposing of organic matter produced in the form of algal blooms, excessive plant growth, undesirable fish and so forth.

4.10 HYDROECOLOGY [HOMS K55]

4.10.1 Introduction

Some of the effects that water resources projects can have on water quality were discussed in 4.9. However, any attempt at defining the quality of a river has to comprise much more than its water quality alone. Indeed, clean waters are a necessary but certainly not sufficient condition to ensure that a river ecosystem is in good ecological health. In
the following discussion, the focus is no longer on water chemistry, under the assumption that there are no water quality problems, and concentrates instead on other aspects of river quality. These are related to the river’s physical structure, habitat availability and biodiversity, the natural processes that determine them and how these aspects may be affected by building and operating water resources projects. The discussion starts by considering the pressing need for adequate environmental management of rivers, as well as its objectives, and defining some basic terms. Then, some fundamental concepts of river morphology and ecology are considered, focusing on the processes more than on the organisms. This is necessary in order to understand the final two sections, which describe the main impacts that water resources projects can have on river ecosystems, as well as the methods commonly applied to reverse or mitigate them.

Hydraulic engineers and hydrologists are the professionals in charge of designing and operating water resources projects and, as such, they should be involved in any interdisciplinary teams responsible for river management and restoration. Reference can be made to Meier (1998a) and WMO/GWP (2006) for short reviews of river ecology, to Jeffries and Mills (1995) and Cushing and Allan (2001) for basic but complete introductions to the topic and to Allan (1995) for an exhaustive review. A good introduction to the subject of river and lake restoration is given by the National Research Council (1992). Cowx and Welcomme (1998), the Federal Interagency Stream Restoration Working Group (FISRWG, 1998), Calow and Petts (1994), Boon and others (1992), and Harper and Ferguson (1995) are compilations that cover in detail most aspects of river management and restoration. Petts and Amoros (1996) provide an integrative vision of ecological change in river systems. Morisawa (1985), Leopold (1994) and Schumm (2005) provide good introductions to river morphology.

This section deals specifically with the ecological impacts of water resources projects on rivers, the main effects on lakes having already been discussed in 4.9.4.

4.10.2 Environmental management of rivers

4.10.2.1 An urgent need

Most rivers in the world have suffered widespread environmental degradation caused by dams, pollution, water diversions, intensive land-use patterns, channelization or river training, flood-plain development, introduction of exotic species and so forth. Owing to these and other human-caused changes, a larger proportion of organisms is extinct or imperilled in freshwaters than in any other type of ecosystem (Angermeier and Karr, 1994), and the economic, ecological, recreational and aesthetic value of many running waters has been sharply reduced. In essence, freshwater ecosystems can be considered to be “biological assets [that are] both disproportionately rich and disproportionately imperilled” (Abramovitz, 1995).

Countering this downward trend in river quality requires sound environmental management. This involves designing and operating new water resources projects in a manner as environmentally friendly as possible to mitigate the impacts of existing, older projects and restore degraded rivers. The main objective of such measures should be to maintain the ecological conditions of healthy, unimpaired rivers and improve them in affected fluvial systems, returning them to higher levels of normality so that they can sustain the full suite of original organisms and habitats, as well as supply goods and services to society.

4.10.2.2 Environmental management objectives

What does it mean to maintain and improve the ecological conditions of a river? Many have understood this to mean to increase productivity and/or biodiversity, but these are very anthropocentric concepts; “more is better” is not really applicable to natural systems. For example, a pristine, ultraoligotrophic alpine lake and its stream outlet have very low concentrations of nutrients and are thus quite sterile environments. Still, they are unimpaired aquatic systems, which cannot be improved. Indeed, improving the lake by making it more productive, for example by fertilizing its waters, would cause eutrophication, with the consequences described in 4.9.5.1. It is clear from the foregoing discussion that the purpose of environmental management of aquatic ecosystems cannot be to increase their productivity.

In simple terms, biodiversity is the variety of organisms and their habitats that can be found in an ecosystem. This concept has more intuitive appeal as an adequate objective for river management and restoration. However, many water bodies have been degraded by the introduction of exotic species; they might have a higher biodiversity but are not better systems for it. It should be clear that the concepts of naturalness and belonging to a place must be involved in the objective of
environmental management; these are explicit in Karr’s (1996) definition of ecological integrity:

... the capacity to support and maintain a balanced, integrated, adaptive ecosystem, having the full range of elements (genes, species, assemblages) and processes expected in the natural habitat of a region...

A river corridor with high ecological integrity should reflect the unimpaired, original conditions in an area, including the presence of all appropriate elements – species, flood-plain ponds and wetlands, for example – and the occurrence of all natural processes such as floods and lateral migration. These conditions should be characterized by little or no influence of human actions – conditions such as those found in national parks. An ecosystem with high integrity reflects natural evolutionary and biogeographic processes (Angermeier and Karr, 1994). Restoring a river to high levels of ecological integrity may be an impossible objective because of economic, social, political or technological constraints. If so, lower levels of integrity must be sought. Some call this intermediate goal rehabilitation or renaturalization; it can also be thought of as partial restoration.

Some rivers have been modified for such a long time, or so intensively, so that little or nothing natural remains about them. Other systems, such as a series of hydropower reservoirs, are or will be continuously managed. These sites cannot be truly restored, eliminating ecological integrity as a management goal. However, one can still strive for ecological health, defined by Karr (1996) as follows:

An ecosystem is healthy when it performs all of its functions normally and properly; it is resilient, able to recover from many stresses, and requires minimal outside care. Ecological health describes the goal for conditions at a site that is managed or otherwise intensively used. Healthy use of a site should not degrade it for future use, or degrade areas beyond the site.

To assess ecological integrity and health, it is necessary to select a benchmark state against which other states can be compared and a variety of measurable ecological indicators. For example, native biodiversity is an important indicator of ecological integrity. Once a restoration goal – a benchmark state – has been selected, the degree of success can be appraised by comparing measured values of the indicators with values for the benchmark. Meier (1998) provides a short summary on the meaning and objectives of river restoration. Karr and Chu (1999) give a basic introduction to the use of multimetric biotic indices to assess ecological integrity and health.

4.10.2.3 The bases for environmental river management

It is clearly much easier, economical and effective to conserve rivers by maintaining their existing ecological integrity than causing undue environmental degradation and then attempting to reverse it with restoration measures.

All aspects of the environmental management of rivers should be based on sound ecological principles. This is easier said than done, as rivers are highly complex natural systems that are structured by many different physical and biological driving forces. Thus, a good understanding of their behaviour requires a strong background in hydrology, hydraulics, fluvial geomorphology and stream ecology. Environmental river management is an interdisciplinary endeavour, generally undertaken by teams composed of engineers, physical scientists and biologists, plus social scientists, economists and managers. Water resources projects can cause varied impacts on the fluvial environment, which cannot be understood and therefore cannot be avoided or mitigated, without some basic knowledge of river behaviour.

Because interdisciplinary river studies are relatively recent, there is still some confusion regarding the most basic definitions. Dunbar and Acreman (2001) define hydroecology as:

the linkage of knowledge from hydrological, hydraulic, geomorphological and biological/ecological sciences to predict the response of freshwater biota and ecosystems to variation of abiotic factors over a range of spatial and temporal scales.

This is precisely the subject of this subsection of the Guide. However, referring to similar issues, Zalewski (2000) defines ecohydrology as:

the study of the functional interrelationships between hydrology and biota at the catchment scale ... a new approach to achieving sustainable management of water.

As Nuttle (2002) points out, however, Zalewski’s definition would imply that ecohydrology is both science and management at the same time. Nuttle correctly notes that a holistic approach to water management depends on the integration of
hydrological and ecological science, but that many other factors, in addition to scientific knowledge, are involved in managing water. He goes on to define ecohydrology as follows:

the sub-discipline shared by the ecological and hydrological sciences that is concerned with the effects of hydrological processes on the distribution, structure, and function of ecosystems, and on the effects of biotic processes on elements of the water cycle.

Both definitions suffer from excessive generality and are too inclusive. Ecohydrology cannot encompass everything that has to do with both water and ecology. Note also that the term ecohydrology has been used in a much more restricted context, having to do with the role of transpiration of terrestrial plants in the global water cycle.

It is therefore preferable to only use the term hydroecology in the sense proposed by Dunbar and Acreman (2001) because it better conveys the fact that it is hydrological and that other physical processes partly drive and structure freshwater ecosystems, but still within a clear understanding that the main focus is on the ecological integrity of these systems. In other words, it is about the effects that hydrology has on the ecology of rivers and lakes, and not vice-versa.

4.10.3 Basic notions of river morphology and ecology

4.10.3.1 The components and extent of fluvial ecosystems

A river ecosystem consists of many interacting organisms of different species, the biota, that live in a physical setting, the abiotic environment. These organisms need food sources in order to stay alive, grow and reproduce, and a place to live in the physical environment: a habitat. They are also subject to mutual biotic interactions, for example, predation (acting either as prey or predator) and competition (fighting for limiting resources, such as space or food).

It is fundamental to emphasize from the beginning that in terms of processes and behaviour, a river comprises much more than the layman’s concept of a wet channel as seen during low-flow periods. In effect, it also incorporates riverbed materials, streambanks and the complete flood plain. The flood plain is the largely horizontal alluvial landform adjacent to a river channel, separated from it by banks. It is constructed by the river from sediment in the present climate and flow regime, and is inundated during moderate flood events. To make this distinction clear, the terms river corridor or river system can be used. Figure II.4.46 shows a

![Figure II.4.46. The concept of a river corridor in the case of a meandering alluvial stream. The river corridor includes all of the landforms shown, as well as the river’s flood plain (Federal Interagency Stream Restoration Working Group, 1998).](image-url)
reach of a meandering river, illustrating some of these features.

Thus, a river corridor has diffuse boundaries with the terrestrial and groundwater systems, the riparian and hyporheic zones, respectively. It includes bars, side arms, flood-plain lakes and all other features created by fluvial processes within the flood plain. These channel and flood-plain features change with time. Therefore, a fluvial ecosystem can be considered to have three spatial dimensions: longitudinal, in the downstream direction, lateral, into the flood plain and vertical, into the alluvial sediments, all of which vary with time (Stanford and others, 1996).

4.10.3.2 Fluvial landforms

The vast majority of river reaches are alluvial, that is, they were formed over unconsolidated sediments that were previously transported and deposited by the stream flow. Non-alluvial rivers are those bounded by bedrock and/or laterally confined by valley walls, so that they are not free to adjust their shape. As shown in Figure II.4.47, an alluvial landscape is determined by the interaction between the hydrological regime or the pattern of flow variability, the sediment load and calibre, the coarse regime of woody debris or tree logs, bed and bank materials and flood-plain vegetation for a given valley slope. Thus, the water, sediment and large woody debris coming into an alluvial reach interact among themselves, and also with the reach bank and bed materials and flood-plain vegetation. By doing so, they continuously modify the river’s movable sediment boundary through erosion and deposition, shaping a dynamic, changing channel, with a given style or pattern. Most rivers are in regime, also referred to as steady state, or dynamic equilibrium, indicating that they are not suffering aggradational or degradational trends. In other words, even though they may keep moving about, their form does not change statistically with time, so that they always look the same.

The currently accepted view among river ecologists, for example, Stanford and others (1996), is that the community structure in flood-prone river systems – the species present in the river corridor – is mainly determined by the dynamics imposed by these physical, hydrogeomorphic processes, not by biotic interaction. The opposite occurs in lakes.

Especially where the local climate and river hydrology allow for perennial flows and the occurrence of woody vegetation, flood-plain corridors of alluvial rivers are among the most dynamic, complex, diverse and productive – as well as endangered – ecosystems on Earth. The number of different species of trees, plants, fishes, invertebrates, birds, mammals and so forth that can live in an intact flood-plain reach such as the one shown in Figure II.4.48 is immense.

**Figure II.4.47. Interaction among the catchment variables that determine the flow, sediment and wood regimes imposed on a reach from upstream. This, in turn, controls its morphological and habitat regimes – the river/flood-plain style and the shifting habitat mosaic, respectively – which together define the ecological regime of the river corridor ecosystem.**
As shown on the left-hand side of Figure II.4.47, the only independent variables in a given river basin, at the longest timescales, are its geology (or physiography: lithology and topography) and climate. The local temperature and rainfall regimes cause weathering of the exposed rocks, determining the character of the soil and the type of vegetation, if any, that can grow within the basin. Together, acting through the stream network, all of these variables prescribe the discharge, sediment and large wood regimes for the reach located downstream. They also drive the load of organic detritus (leaves, twigs and organic silt, commonly referred to as particulate organic matter), dissolved matter fluxes such as solutes and the stream temperature regime. Human influences, including land-use changes, dam building and flood control measures, have dramatically altered all of these natural regimes in many rivers of the world.

The study and management of alluvial rivers require an understanding of their variability in space and time, involving the following three considerations (Schumm, 2005):

(a) There is a continuous spectrum of river types or styles: meandering, single-thread sinuous, wandering or braided. These styles depend mainly on the flow, flood sediment load and size, regimes, geologic history (particularly the valley slope), vegetation and the occurrence of previous conditioning events. Different styles of river employ different mechanisms to build and interact with their flood plains according to various hydrological and geomorphological processes, resulting in distinct patterns of temporal and spatial morphological variability, both at the surface of the river corridor and below it, within the alluvial aquifer;

(b) Rivers change over longer timescales as a result of climate or hydrological variability;

(c) There can be a considerable amount of variability within any given reach as a result of local geomorphic and geological controls such as tributaries, bank material variability and vegetation.

Over time, driven by the flow regime, mostly by periodic flooding, the channel moves across the valley floor, reworking the bed and flood-plain sediments, thus destroying but also creating side arms, wetlands, ponds and a host of other riverine landscape features, which are quickly colonized by riparian vegetation. Figures II.4.46 and II.4.48 show such processes acting in an alluvial river corridor, in this case for streams with a meandering style. In this manner, the fluvial processes of erosion and sedimentation, interacting with vegetation growth, continuously modify not only the main wet channel but indeed the entire river corridor, even though, from a distance, the landscape might seem unchanged, because it is in regime. This simple fact explains why changes in the flow, sediment and large wood regimes, often caused by water resources projects, can cause wide-reaching impacts in the downstream river corridor ecosystems: they are

Figure II.4.48. Pristine river corridor of the Palena river in Chilean Patagonia, an active meandering single-thread sinuous system of high ecological integrity. Note the diversity of forms, water depths and velocities, ages of vegetation stands and the abundance of large woody debris in the channel. Water is off colour because of glacial melt contributions. The patch of younger vegetation seen on the meander point bar at the right of the picture is indicated with an arrow.
alterations in the three main ingredients of a river's functioning and, as such, should cause a change in the resulting landscape regime.

As mentioned above, rivers of different style move about and create their flood plains through different mechanisms. For example, meandering rivers migrate laterally by eroding the existing flood-plain material on the outer side of bends and at the same time depositing sediment on the point bar formed on the inner side, a process known as lateral accretion; they tend to create oxbow flood-plain lakes (see Figures II.4.46 and II.4.48). In contrast, wandering gravel-bed rivers create mid-channel sediment bars, which can be colonized by vegetation. During high discharges, the vegetation traps fine sediment, thus raising the surface of the bar by vertical accretion until it becomes an established island, which later becomes part of the flood plain when the river abandons one of its adjacent side channels. Bars split the flow, thus creating multiple channels or anabranches.

River ecologists have found clear relationships between river type or pattern and some important ecological indicators of ecosystem health, such as habitat complexity and biodiversity (see Figure II.4.49). This is why it is so important in river management and restoration to consider the relationship between hydrogeomorphic processes and river styles.

4.10.3.3 River morphology drives river ecology

As previously noted, in unaltered river systems, hydrogeomorphic processes create a complex environment, which is highly heterogeneous, both spatially and temporally. This changing mosaic of in-channel, flood-plain and hyporheic or underground habitat patches provides sustenance and a place to live for many different species of plants and animals, both aquatic and riparian, whose life cycles have evolved in response to the highly dynamic and heterogeneous environment (Stanford and others, 1996). Thus, the river or flood-plain style can be considered to be a geomorphological template, determining not only the plan form of the channel, but also the riverine landscape dynamics within the flood plain, thereby structuring the habitat template available to organisms. Hydrology and morphology interact, setting the stage for riparian vegetation to colonize and driving the river corridor ecology through the establishment of a shifting habitat mosaic.

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Figure II.4.49. Continuum of river styles associated with increasing stream power, indicating the varying rate of change for the shifting habitat mosaic, as well as some relevant hydrogeomorphic and biogeomorphic processes (simplified from Lorang and Hauer, 2006)
Intact river corridors are highly diverse and include a suite of aquatic, riparian and hyporheic habitat patches. The habitat diversity in a reach depends on the river or floodplain style, which determines the rate at which habitat patches change in space and time. The spatial variability is reflected in the availability of deep and shallow waters, sandy and gravelly underground environments, old forest and young vegetation patches, fast and slow currents, cold or clear springbrooks and warmer or more turbid main channels, dry gravel bars and silty wetlands and the like. The temporal variability of fluvial habitats is tied to different timescales: diel (day-night) cycles affect water temperature; at the seasonal scale, flooding cycles result in inundation of the flood plain and reconnection of the river with its lateral aquatic habitats (ponds, wetlands, side arms), while deciduous trees shed their leaves, contributing a pulse of organic detritus; at geomorphological timescales, which depend on river style, landforms are created and destroyed, thus reshaping the habitat mosaic. Note that the concept of a shifting habitat mosaic implicitly incorporates both the temporal (shifting) and spatial (mosaic) variability inherent to natural river systems.

Ecologists have shown that the greater the habitat complexity in a river corridor, the greater the biodiversity that it can sustain. Indeed, in order to survive, grow, and reproduce, organisms need food and a place to live—a habitat—within the physical environment they inhabit. Not only are these requirements particular to each species, but a given species can have different dietary and habitat needs at different stages of life, for example, a brown trout egg, fry and adult; or a nesting, versus a juvenile duck. The key here is that a certain river reach must supply the whole range of habitat needs for a species to permanently reside in it. This explains why diverse, complex environments are able to sustain a much higher diversity of organisms than uniform environments.

Organisms of most species not only have varying habitat requirements as they age, but also at different times of the day or during seasonal cycles. This implies that individuals must be able to move between different habitat patches. They might move once in a lifetime, along the longitudinal dimension of the river, as is the case with some species of salmon migrating from the ocean towards headwaters, or on a daily basis, for example, when an individual switches between a feeding position in a riffle and its resting position under an undercut bank. Movement can take place along the transversal or lateral dimension, for example when fish species use lateral habitats to spawn, as shown in Figure II.4.50. Many species have patchy spatial distributions, with few individuals per population. Movement between linked patches, which requires high connectivity, is keeping such species from becoming locally extinct.

4.10.4 Ecological impacts of water resources projects

4.10.4.1 The importance of change, heterogeneity and connectivity

Change created by the disturbance regime is a fundamental component of a healthy river ecosystem that needs to be maintained when deciding on the design and operation of water resources projects, or reinstated if one wants to mitigate the impact of existing works. Indeed, in many cases, excellent restoration results can be achieved by simply removing the impact-causing factors: reinstating the original flow, sediment and wood regimes to the river without the need for further manipulation.

If a highly diverse river corridor is not allowed to change, for example by preventing the occurrence of floods or by separating the flood plain from the main channel by embankments, old patches will no longer be destroyed and new habitats will not be created, impeding the recruitment of seedlings. This will result in progressive aging of the flood-plain forests as the existing vegetation stands mature and take over the originally heterogeneous fluvial landscape. The final result will be a uniform river corridor, which sustains a lower biodiversity. This example illustrates that not only the river’s morphology, but also its habitat availability can be considered to

![Figure II.4.50. Use of spawning habitat by different fish species in the wandering upper Rhone river, France (Roux and Copp, 1996)](image)
be in a state of regime, or dynamic equilibrium, where individual patches are continuously changing. However, the overall availability of different habitat types remains more or less the same over a reach. The conceptual model of the regime behaviour of habitat in a river corridor was described earlier as the shifting habitat mosaic.

Water resources projects tend to stabilize, oversimplify and disconnect river corridors, resulting in spatially homogeneous conditions that are unable to provide varied habitat features for a diverse range of species. For example, rivers are channelized between levees, resulting in uniform trapezoidal cross-sections, with no variability in depths or velocities, and in severed connections between the main channel and the flood-plain features that are left dry behind the dykes when the river floods. The alluvial reach shown in Figure II.4.50 shows a variety of aquatic environments used for spawning by a wide range of fish species. If this reach were to be channelized for navigation, it would end up as a uniformly deep, narrow, single-thread channel, with much reduced heterogeneity. Many habitats would be lost in the lateral channels and flood plain, resulting in a sharp decrease in fish biodiversity.

Unfortunately, hydraulic works generally affect a river’s temporal dynamics, too, impeding change. For example, when floods are regulated by dams, hydrogeomorphic processes can no longer rework the flood-plain sediments, with the aforementioned consequences.

In high-energy fluvial systems that have large slopes and/or floods in relation to their sediment size, the disturbance pattern, that is, the rate at which habitats are created and destroyed, may be too fast to allow for high biodiversity. Typical examples are gravel and sand-bed braided rivers, where bars and islands have such a high turnover rate that most of the in-channel habitat patches are relatively recent or young. In contrast, a low-energy meandering river, which migrates laterally at a slow rate, might have a large proportion of its flood plain under mature vegetation, with little availability for young patches. Such a system will also be too homogeneous to sustain a large biodiversity. Indeed, it has been hypothesized that diversity is maximized in fluvial ecosystems with an intermediate rate of disturbance: those with intermediate energy. This corresponds to river styles between single-thread sinuous and wandering, that show a low tendency to braiding, a high level of anabranching or formation of multiple channels, moderate meandering, high retention of large woody debris and a strong tendency to form bars, as shown in Figure II.4.49. In other words, too much change – continuously resetting the system, or too little change – allowing one type of habitat to dominate over all others – results in decreased diversity.

Therefore, clean water is only one of many ingredients of a healthy river ecosystem.

4.10.4.2 Some recurrent impacts

Most water resources projects include channelization or river training, regulation and diversion, altering the flow, sediment and wood regimes and cutting off the ecological connectivity along a river’s spatial dimensions, thus decreasing the ecological integrity of the fluvial system. The effects of other human-induced changes, such as point pollution and overfishing, are generally reversible and the mitigation strategies are obvious, except for extinction which is rarely reversible. Definitive solutions are hard or even impossible to achieve in the cases of diffuse or non-point pollution and the invasion by exotic species, such as the introduction of the parasitic sea lamprey in the Great Lakes watersheds.

The effects of dams, diversions, and channelization works show recurrent patterns worldwide (Stanford and others, 1996; Brookes, 1988; Petts, 1984). These include the following effects:

(a) Habitat diversity and connectivity are reduced:
Flow, sediment and large wood regimes are altered, affecting the fluvial dynamics that create heterogeneous in-channel and flood-plain habitat patches. The longitudinal connectivity is interrupted by dam barriers leading to fish passage problems, for example. Seasonal flow variability is reduced, but hourly or daily discharges can fluctuate wildly. The natural temperature regime is lost because of hypolimnetic releases. Channelization procedures disconnect the wetted channel from its flood plain, altering baseflow or groundwater interaction, degrading riparian habitats, impeding seasonal flood-plain inundation – and thus reconnection to the channel – and creating an homogeneous wetted channel. Dewatering severs the longitudinal dimension and can cause high mortality of aquatic organisms through stranding. The lack of flooding allows vegetation to encroach upon the channel and then mature, resulting in less diverse riparian zones. In short, hydraulic works create discontinuities along the river’s spatial dimensions and homogenize channel and flood-plain habitat conditions;
(b) Native diversity decreases while exotic species proliferate: The altered hydrological, sediment and temperature regimes do not provide adequate environmental conditions for most native species. However, the homogenization of habitats allows exotic species to compete better. For example, in the United States, the native Colorado river fish species were adapted to extreme turbidities, flows and temperature regimes. Because of their adaptation, they fared well where no exotic species could survive. When dams were built, however, they regulated the flow conditions and started releasing cold, clear hypolimnetic waters. As a result, the non-native rainbow trout was able to invade and outcompete the native species, driving them to the brink of extinction.

Ecosystem productivity can often be enhanced by the changes, for example when a highly variable flow regime is regulated into a constant discharge year-round, or when dams release clear, nutrient-laden waters from the bottom of reservoirs. In this case, a handful of species can reach large populations, but this is always matched by a decrease in diversity owing to the extinction of many other, rarer, species that depended for their survival on the temporal variability of the flows and the associated spatial variability of the habitat.

The ecological impact of water resources projects is not always predictable quantitatively because the relationship between hydrology, morphology and ecology, namely hydroecology, is not at all simple. Certain impacts can be mitigated if the right design and operational procedures are adopted (see Petts, 1984; Brookes, 1988; Gore and Petts, 1989; Gardiner, 1991; National Research Council, 1992; Cowx and Welcomme, 1998). For example, selective multidepth withdrawal structures can alleviate water quality problems and help maintain the original temperature regime downstream of dams. Difficult societal and economic decisions can be involved, as is the case when a complete flow regime, including extremes of floodplain inundation and low-flow periods, must be determined or when it is desirable to allow lateral migration of a river in order to re-establish a shifting habitat mosaic.

4.10.5 Mitigation of ecological impacts

The most important conclusion of the above summary of hydroecology is as follows: ecologically healthy river corridors are very complex landscapes that depend on continued change to maintain their shifting habitat mosaic and connectivity, and thus their natural communities. Continued change is produced by hydrogeomorphic forces linked to flooding disturbances.

However, most of what is generally referred to as the environmental management of rivers does not centre around these fundamental scientific concepts and how they can be used to attempt effective conservation or restoration of rivers. Instead, it focuses constantly on two technical aspects which, in light of the complexity of river corridor systems, are of relatively minor importance, namely the restoration of the physical habitat by placing structures in rivers, and the determination of minimum instream flows.

Most of these approaches have ignored some of the basic principles of river behaviour. Restoring habitat by locating fixed structures in a channel goes against the natural tendency of a river to move about. Habitat is essentially dynamic – shifting, not fixed. As a river changes, it manufactures habitat. Also, this type of habitat enhancement technique is generally geared exclusively towards fish. Of course, such knowledge can still be useful when rehabilitating streams that cannot be allowed to migrate laterally, for instance in urban settings. Examples of and further references to this approach can be found in Cowx and Welcomme (1998).

Minimum instream flows (see 4.6.2.3.5), which in many countries are called ecological or environmental flows, have not generally led to much more than what the name implies: a minimal, year-round constant flow to maintain a semblance of an aquatic ecosystem. However, there is certainly scope for these methodologies to consider some of the aspects that have been described previously as being fundamental for a river system to maintain or recover a high level of ecological health or integrity. In fact, as instream flow methodologies set levels of flow considered to be adequate for different purposes, if the right purposes are taken into account and the models based on theory or field data can represent the relationships with flow, good results can be expected.

One of the main problems is that most instream flow models were based on the end results of the causal chain, rather than the processes that created habitat in the first place. Until the late 1990s, there were a variety of approaches in use. Hydrological methodologies prescribed simple percentages or more complicated functions of the available flow. Hydraulic methods attempted to preserve a proportion of the available wet habitat based on concepts of marginality. Habitat models computed available
habitat for a certain life stage of a given species based on habitat suitability criteria. Hydraulic modelling of a reach under varying flows was yet another approach. Jowett (1997) compares them and offers detailed references.

The building block methodology (Tharme and King, 1998) was the first of a new series of instream flow models that have been labelled holistic methodologies in that they address the flow needs of the entire riverine ecosystem, based on explicit links between changes in hydrological regime and the consequences for the biophysical environment. This approach uses findings from biological studies in order to recommend levels of flow to meet various ecological criteria during the year, such as connectivity with lateral spawning habitats and fish migrations. The different flows needed during specific months or seasons of the year are then used as building blocks to form the overall instream flow hydrograph. Interannual variability can also be added by specifying instream flow hydrographs for dry, average and wet years. Tharme (2003) presents a global assessment of instream flow methodologies, comparing holistic approaches with the previous three types and giving an inclusive listing, with references, of the 207 individual techniques developed at the time.

Holistic methodologies are clearly top-down approaches, unlike habitat models, which by definition are bottom-up or reductionistic. If the hydrological, hydraulic and habitat models can be said to focus on the lowest levels of the ecological chain, the symptoms, holistic methodologies can be seen as focusing on the intermediate levels of causality. Caution must be exercised simply because flows do not explain all of the ecological variance in a river reach. As previously noted, the interaction between flow, sediment and wood regimes, as well as reach materials and vegetation, determine the river’s morphologic style, and thus the shifting habitat mosaic. The temperature regime is also relevant, as aquatic organisms are mostly ectotherms. Thus, any application of a holistic instream flow methodology in a degraded reach must also consider the restitution of more natural sediment, wood and temperature patterns.

Since the fundamental process attribute of a river ecosystem is the shifting habitat mosaic – which depends on the geomorphic template given by the river style – it should be possible to use instream flow methodologies to focus on these causative mechanisms, rather than looking farther along the causal chain. While it must be recognized that river style and the subsequent shifting habitat mosaic might be determined mainly by flows above a certain threshold and organisms in the reach need to survive there year round, some work is indeed being undertaken in that direction. For example, Lorang and others (2005) used remotely sensed imagery to evaluate geomorphic work in a distributed fashion – pixel by pixel – across a flood-plain reach of a gravel-bed river. They did this over a range of flows, carrying out data-based modelling of stream power at each pixel, as a function of flow. Coupling this type of research with sediment transport and hydraulic models could lead to hydroecological methods that would assess the magnitude and duration of the flows needed to perform sufficient work in order to maintain the river style and shifting habitat mosaic in a river reach.

This offers exciting possibilities for the future because geomorphic work requires the integration of variables that can be obtained by combining magnitude and duration in different ways, thus providing the much sought-after flexibility. Also, such types of result could easily be added as an extra building block into holistic instream flow methodologies, ensuring the maintenance of the main drivers of ecological health and integrity in rivers.

References and further reading


CHAPTER 4. APPLICATIONS TO WATER MANAGEMENT

Rome, Food and Agriculture Organization of the United Nations.


