Guide to Hydrological Practices

Volume II

Management of Water Resources and Application of Hydrological Practices

World Meteorological Organization
Weather • Climate • Water

WMO-No. 168

Weather • Climate • Water

Guide to Hydrological Practices

Volume II
Management of Water Resources and Application of Hydrological Practices

WMO-No. 168

Sixth edition

2009

World Meteorological Organization
Weather • Climate • Water

WMO-No. 168

Chairperson, Publications Board
World Meteorological Organization (WMO)
7 bis, avenue de la Paix
P.O. Box 2300
CH-1211 Geneva 2, Switzerland

Tel.: +41 (0) 22 730 84 03
Fax: +41 (0) 22 730 80 40
E-mail: publications@wmo.int

ISBN 978-92-63-10168-6

VOLUME II

TABLE FOR NOTING SUPPLEMENTS RECEIVED

Supplement No.	Dated	Inserted	
		by	date
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			
16			
17			
18			
19			
20			
21			
22			
23			
24			
25			
26			
27			
28			
29			
30			

CONTENTS

		Page
PREFACE		**xi**
ACKNOWLEDGEMENTS		**xiii**
CHAPTER 1 – INTRODUCTION		**II.1-1**
1.1	Background	II.1-1
1.2	Scope	II.1-1
1.3	Contents of the Guide	II.1-2
1.4	The Hydrological Operational Multipurpose System	II.1-4
References and further reading		II.1-4
CHAPTER 2 – HYDROLOGICAL SERVICES		**II.2-1**
2.1	Introduction	II.2-1
2.2	Responsibilities and functions of Hydrological Services	II.2-1
	2.2.1 Nature of the products and services of a Hydrological Service	II.2-1
	2.2.2 Clients of hydrological products and services	II.2-2
	2.2.3 Managing relationships with clients	II.2-3
	2.2.4 Hydrological products and services	II.2-3
	2.2.5 Functions and activities of a Hydrological Service	II.2-3
	2.2.6 Evaluation of products and services, and quality management	II.2-4
	2.2.7 Legal basis for operations and organizational arrangements	II.2-5
	2.2.8 Managing relationships with other institutions	II.2-6
	2.2.9 Data exchange	II.2-7
2.3	Planning and strategy	II.2-8
2.4	Human resources management and capacity-building	II.2-9
	2.4.1 Management	II.2-9
	2.4.2 Training and continuing education	II.2-10
2.5	Financial and asset management	II.2-11
	2.5.1 Sources of revenue	II.2-11
	2.5.2 Budgeting and monitoring financial performance	II.2-12
	2.5.3 Asset management	II.2-12
	2.5.4 Database security	II.2-12
References and further reading		II.2-13
CHAPTER 3 – INTEGRATED WATER RESOURCES MANAGEMENT		**II.3-1**
3.1	Introduction	II.3-1
	3.1.1 Sustainable water development	II.3-1
	3.1.2 The changing nature of the resource	II.3-1
	3.1.3 Changing attitudes to management	II.3-2
3.2	Integrated water resources management	II.3-3
3.3	Rationale for integrated water resources management	II.3-4
	3.3.1 Water quantity and quality	II.3-4
	3.3.2 Surface water and groundwater	II.3-4
	3.3.3 Upstream and downstream considerations	II.3-4
	3.3.4 Water, land and other resource systems	II.3-5
	3.3.5 Environment, the economy and society	II.3-5
	3.3.6 Vertical and horizontal fragmentation: systems and silos	II.3-5
	3.3.7 Collaboration, coordination and coherence	II.3-6

Page

3.4 Evolution of integrated water resources management II.3-7
3.4.1 United States of America: Ohio conservancy districts, Tennessee Valley Authority II.3-7
3.4.2 Canada: Conservation authorities II.3-7
3.4.3 United States and Canada: the great lakes II.3-7
3.4.4 Australia: total catchment management II.3-8
3.4.5 New Zealand: Resource Management Act II.3-8
3.4.6 South Africa II.3-8

3.5 Perspectives on integrated water resources management II.3-9
3.5.1 Dublin Conference: Earth Summit, 1992 ..... II.3-9
3.5.2 World Water Council and the World Water Fora II.3-10
3.5.3 Global Water Partnership II.3-10
3.5.4 World Summit on Sustainable Development, Johannesburg, 2002 ..... II.3-11

3.6 Elements of best practice for integrated water resources management II.3-11
3.6.1 Alternative interpretations: comprehensive versus integrated approaches II.3-11
3.6.2 Vision for a desirable future II.3-12
3.6.3 Spatial scale: watershed, subwatershed, tributary and site II.3-12
3.6.4 Partnerships and alliances II.3-13
3.6.5 Links to regional planning and impact assessment II.3-13
3.6.6 Designing institutional arrangements II.3-14
3.6.7 Monitoring and evaluating II.3-15

3.7 Cautions regarding integrated water resources management II.3-15
3.7.1 When to apply integrated water resources management II.3-15
3.7.2 Implementation gap II.3-15

References and further reading II.3-15

CHAPTER 4 – APPLICATIONS TO WATER MANAGEMENT II.4-1

4.1 Water resource assessment and water projects II.4-1
4.1.1 The need for water resource assessment II.4-1
4.1.2 Water resource assessment programme components II.4-1
4.1.3 Evaluation of water resource assessment activities II.4-3
4.1.4 Water projects II.4-3
4.1.5 Purposes served by a water management project II.4-3
4.1.6 Multi-purpose projects II.4-4
4.1.7 Project cycle II.4-4
4.1.8 Preliminary investigation of water management projects II.4-5

4.2 Estimating reservoir capacity and yield II.4-7
4.2.1 General II.4-7
4.2.2 Concepts of yield II.4-8
4.2.3 Estimation of storage–yield relationships II.4-11
4.2.4 Classifications of yield II.4-12
4.2.5 Probabilistic approach II.4-13
4.2.6 Multi-purpose reservoirs and operating rules II.4-16
4.2.7 Multi-reservoir water resource systems II.4-17
4.2.8 Incidental effects of reservoirs II.4-18
4.2.9 Remote-sensing estimates of reservoir capacity II.4-19
4.2.10 Climate change II.4-20

4.3 Flood management II.4-20
4.3.1 General II.4-20
4.3.2 Flood management strategies II.4-20
4.3.3 Integrated flood management II.4-21
4.3.4 Structural measures II.4-22
4.3.5 Non-structural measures II.4-27
4.3.6 Flood emergency management II.4-29

4.4 Irrigation and drainage II.4-30
4.4.1 Irrigation II.4-30

Page

4.4.2 Agricultural drainage II.4-39
4.4.3 Use of remote-sensing and general information systems in irrigation and drainage .. II.4-42

4.5 Hydropower and energy-related projects II.4-42
4.5.1 General II.4-42
4.5.2 Hydropower II.4-43
4.5.3 Operation of a hydroelectric system II.4-51
4.5.4 Other projects related to energy production II.4-52

4.6 Navigation and river training II.4-54
4.6.1 Application of hydrology to navigation II.4-54
4.6.2 Classification of river training II.4-59
4.6.3 Erosive forces due to channel flow II.4-60
4.6.4 Erosive forces caused by waves and craft II.4-62
4.6.5 Evolution and characterization of river bends II.4-62
4.6.6 Determination of design discharges and stages II.4-63

4.7 Urban water resources management II.4-64
4.7.1 General II.4-64
4.7.2 Urban development impacts II.4-65
4.7.3 Urban storm drainage design II.4-66
4.7.4 Urban drainage control measures II.4-71
4.7.5 Urban drainage management II.4-71
4.7.6 Remote-sensing estimates for land use II.4-73

4.8 Sediment transport and river channel morphology II.4-73
4.8.1 General II.4-73
4.8.2 Catchment erosion II.4-73
4.8.3 Channel erosion II.4-73
4.8.4 River systems II.4-73
4.8.5 Flow regimes and bed forms II.4-75
4.8.6 Transportation of sediments in channels II.4-77
4.8.7 Sedimentation II.4-80
4.8.8 Sediment control measures II.4-81

4.9 Water quality and the conservation of aquatic ecosystems II.4-81
4.9.1 General II.4-81
4.9.2 Relationships between water quantity and water quality II.4-82
4.9.3 Effects of water resources projects on water quality in streams and rivers II.4-83
4.9.4 Effects of water resources projects on water quality in large lakes and reservoirs II.4-84
4.9.5 Water quality changes caused by pollution II.4-84
4.9.6 Measures to reduce effects of pollution on water quality II.4-86

4.10 Hydroecology II.4-86
4.10.1 Introduction II.4-86
4.10.2 Environmental management of rivers II.4-87
4.10.3 Basic notions of river morphology and ecology II.4-89
4.10.4 Ecological impacts of water resources projects II.4-93
4.10.5 Mitigation of ecological impacts II.4-95

References and further reading II.4-96

CHAPTER 5 – EXTREME VALUE ANALYSIS II.5-1

5.1 Introduction II.5-1

5.2 Statistical series and return periods II.5-1

5.3 Probability distributions used in hydrology II.5-3
5.3.1 Normal family: N, LN and LN3 .... II.5-4
5.3.2 Extreme value distributions: Gumbel, generalized extreme value and Weibull II.5-4
5.3.3 Pearson type III family II.5-6
5.3.4 Halphen family: types A, B and type B^{-1} II.5-6
5.3.5 Generalized logistic distribution II.5-7

Page

	5.3.6	Generalized Pareto distribution	II.5-7
	5.3.7	Non-parametric density estimation method	II.5-7
5.4	Hypothesis testing		II.5-7
	5.4.1	Wald–Wolfowitz test for independence and stationarity	II.5-8
	5.4.2	Mann–Kendall test for trend detection	II.5-9
	5.4.3	Mann–Whitney test for homogeneity and stationarity (jumps)	II.5-9
	5.4.4	Sample size and length of record	II.5-10
	5.4.5	Grubbs and Beck test for detection of outliers	II.5-10
	5.4.6	Bayesian procedures	II.5-10
5.5	Population statistics and parameter estimation		II.5-11
	5.5.1	Parameter calculation methods	II.5-11
	5.5.2	Use of logarithmic transformations	II.5-12
	5.5.3	Historical Information	II.5-12
	5.5.4	Record augmentation	II.5-12
	5.5.5	Analysis of mixed populations	II.5-13
	5.5.6	Frequency analysis and zeroes	II.5-13
5.6	Probability plots and goodness-of-fit tests		II.5-14
	5.6.1	Plotting positions and probability plot	II.5-14
	5.6.2	Goodness-of-fit tests	II.5-14
	5.6.3	Information criteria	II.5-14
5.7	Rainfall frequency analysis		II.5-15
	5.7.1	Assessment of rainfall data for frequency analysis	II.5-15
	5.7.2	At-site frequency analysis of rainfall	II.5-16
	5.7.3	Regional rainfall frequency analysis	II.5-18
	5.7.4	Frequency analysis of area-averaged rainfall	II.5-19
	5.7.5	Storm rainfall analysis for hydrological design applications	II.5-20
5.8	Low-flow analyses		II.5-31
	5.8.1	General	II.5-31
	5.8.2	At-site low-flow frequency analysis	II.5-32
	5.8.3	Low-flow frequency estimation at partial-record sites using base-flow measurements..	II.5-34
	5.8.4	Regionalization of low-flow frequency statistics	II.5-35
	5.8.5	Flow-duration curves	II.5-36
5.9	Frequency analysis of flood flows		II.5-38
	5.9.1	Regionalization of flood flows	II.5-38
	5.9.2	Homogeneous region delineation	II.5-39
	5.9.3	Regional flood estimation methods	II.5-40
	5.9.4	At-site and regional flow–duration–frequency approach	II.5-42
	5.9.5	Combination of single-site and regional data	II.5-42
	5.9.6	Flood frequency analysis and climate variability	II.5-43
5.10	Estimating design floods		II.5-44
	5.10.1	General	II.5-44
	5.10.2	Design floods	II.5-45
	5.10.3	Data preparation	II.5-47
	5.10.4	Design flood computation techniques	II.5-48
	5.10.5	Flood hydrograph conceptual models	II.5-51
	5.10.6	Snowmelt contribution to flood	II.5-52
	5.10.7	Calculating discharges from urban drainage systems	II.5-52
	5.10.8	Risk	II.5-52
References and further reading			II.5-53
CHAPTER 6 – MODELLING OF HYDROLOGICAL SYSTEMS			**II.6-1**
6.1	Mathematical deterministic models		II.6-1
	6.1.1	Black box models	II.6-2
	6.1.2	Artificial neural networks	II.6-3

Page

6.1.3 Conceptual models II.6-4
6.1.4 Distributed models II.6-8
6.1.5 Parameter evaluation II.6-10
6.1.6 Selection of models II.6-11

6.2 Time series and spatial analysis II.6-12
6.2.1 Stochastic simulation of hydrological time series II.6-13
6.2.2 Change detection in hydrological records II.6-14
6.2.3 Spatial analysis in hydrology II.6-17

6.3 Modelling hydrological systems and processes II.6-18
6.3.1 Introduction II.6-18
6.3.2 Rainfall–runoff relationships II.6-19
6.3.3 Groundwater modelling II.6-25
6.3.4 Snowmelt models II.6-30
6.3.5 Streamflow routing II.6-39
6.3.6 Modelling other processes II.6-42

6.4 Modelling challenges II.6-48
6.4.1 Accuracy and availability of input data II.6-48
6.4.2 Ungauged basins II.6-48
6.4.3 Coupling of models II.6-49

References and further reading II.6-50

CHAPTER 7 – HYDROLOGICAL FORECASTING II.7-1

7.1 Introduction to hydrological forecasting II-7-1
7.1.1 Scope II-7-1
7.1.2 Hydrological forecast operations II-7-1
7.1.3 End-to-end hydrological forecasting systems II-7-2
7.1.4 Uncertainty and probabilistic forecasts II-7-3
7.1.5 Dissemination of forecasts and warnings II-7-4
7.1.6 Decision support II-7-6
7.1.7 Cooperation with the National Meteorological Service II-7-7

7.2 Data requirements for hydrological forecasts II-7-7
7.2.1 General II-7-7
7.2.2 Data required to establish a forecasting system II-7-8
7.2.3 Data required for operational purposes II-7-9

7.3 Forecasting techniques II-7-10
7.3.1 Requirements for flood forecasting models II-7-10
7.3.2 Flood forecasting methods II-7-11
7.3.3 Model updating techniques II-7-14
7.3.4 Forecast verification II-7-15

7.4 Forecasting flash floods II-7-16
7.4.1 National flash flood programmes II-7-16
7.4.2 Local flash flood systems II-7-17
7.4.3 Wide-area flash flood forecasts II-7-19
7.4.4 Flash flood guidance II-7-19
7.4.5 Dam-break flash flood forecasting II-7-21
7.4.6 Storm surges in rivers II-7-21
7.4.7 Urban flooding II-7-22
7.4.8 Flooding from local drainage II-7-22

7.5 Long-term forecasting II-7-23
7.5.1 Water supply forecasting II-7-23
7.5.2 Flow recession forecasting II-7-24
7.5.3 Time-series analysis II-7-25

7.6 Snowmelt forecasts II-7-25
7.6.1 General II-7-25

Page

7.6.2 Snowmelt runoff processes in lowland and mountain rivers II-7-25
7.6.3 Short- and medium-term snowmelt runoff forecasts II-7-26
7.6.4 Long-term snowmelt forecasts II-7-26

7.7 Forecasts of ice formation and break-up II-7-27
7.7.1 General II-7-27
7.7.2 Long-term ice forecasts II-7-28
7.7.3 Ice jams and methods of forecasting high water levels II-7-28

References and further reading II-7-31

PREFACE

In September 2000, world leaders agreed to the United Nations Millennium Declaration, from which was soon derived a set of eight time-bound and measurable goals and targets for combating poverty, hunger, disease, illiteracy, environmental degradation and gender inequality. These eight objectives are known as the United Nations Millennium Development Goals (MDGs). The attainment of each of these Goals depends, to a great extent, on the availability of appropriate freshwater and on the protection of the population from the ravages of flooding. This, in turn, places a major responsibility on the National Hydrological and Hydrometeorological Services to support the necessary actions at the national level, in the face of ever-increasing demands on the limited freshwater resources available to WMO Members. In transboundary basins, in particular, where concerns are often driven by the need for equitable distribution of these limited resources, appropriate operational mechanisms to share them may have to be established and maintained among the relevant riparian countries.

One of the purposes of the World Meteorological Organization (WMO) is to promote the standardization of meteorological and hydrological observations and to ensure uniform publication of observations and statistics. With this objective, the World Meteorological Congress has traditionally adopted Technical Regulations laying down the meteorological and hydrological practices and procedures to be followed by Members of the Organization. These *Technical Regulations* (WMO-No. 49) are supplemented by a number of manuals and guides describing in more detail the practices and procedures that Members are requested or invited to follow, respectively, in monitoring and assessing their respective water resources. It is therefore hoped that improved uniformity and standardization in hydrological practices and procedures will also contribute to enhanced collaboration among WMO Members and further facilitate regional and international cooperation.

The aim of the *Guide to Hydrological Practices* is to provide the relevant information on current practices, procedures and instrumentation to all those engaged in the field of hydrology, thereby enabling them to carry out their work more successfully. Complete descriptions of the theoretical bases and the range of applications of hydrological methods and techniques are beyond the scope of this guide, although references to such documentation are provided wherever applicable. Detailed procedures for monitoring hydrological parameters are dealt with in the specific WMO manuals.

It is hoped that this guide will be of use, not only to Members' National Services, but also to various other stakeholders and agencies involved in water resources management in general, and in water resources monitoring and assessment in particular. The WMO Commission for Hydrology (CHy) has therefore decided to make this guide a "living" document, which will be updated periodically and posted on the Internet. This Guide will also represent one of the building blocks of the WMO Quality Management Framework – Hydrology, which is currently being developed in order to support Members and their National Services by ensuring that the activities they undertake, such as hydrological data acquisition or delivery of services and products, are indeed performed efficiently and effectively. Users of the Guide are therefore invited to continue providing their comments and suggestions for its further improvement.

The *Guide to Hydrological Practices* is published in English, French, Russian and Spanish. However, as with previous versions, several Members have announced their intention to translate this Guide into their national languages.

It is a pleasure to express my gratitude to the WMO Commission for Hydrology for taking the initiative to oversee the revision of the *Guide to Hydrological Practices*.

(M. Jarraud)
Secretary-General

ACKNOWLEDGEMENTS

Following the expressed needs of its members, the Commission for Hydrology decided to update and publish this, the sixth edition of the *Guide to Hydrological Practices* (Guide). This decision was made based on comments and experiences in using the fifth edition of the Guide and recognizing its great value to the National Hydrological Services and professionals working in water-related fields. More than 40 seasoned experts from around the world have contributed to the preparation of this edition. As a result, it is oriented towards practical applications and to fit within a quality management framework that is being initiated by the Commission for Hydrology. It is with great pleasure that I express the gratitude of the Commission to those experts who volunteered to be involved in the material compilation and preparation process, and enabled this enormous task to be accomplished.

I extend my deep appreciation to the members of the Review Committee established by the Commission for Hydrology that has overseen the revision of the Guide. The Review Committee headed by Karl Hofius (Germany), and consisting of Suresh Chandra (India), Denis Hughes (South Africa), Fred Kyosingira (Uganda), Paul Pilon (Canada), Marco Polo Rivero (Venezuela) and Avinash Tyagi (Director, Climate and Water Resources Department, World Meteorological Organization (WMO), was instrumental in identifying the areas in the fifth edition that required revision and updating, identifying the experts responsible for redrafting and peer review of various chapters and sections, and carrying out the review of the experts' contributions.

I express my sincere thanks and recognition to the experts who contributed to the redrafting and revision of the Guide. The following experts contributed to the updating and revision of the chapters (indicated in brackets) of Volume I of the Guide: Svein Harsten (Chapters 2 and 5); Robert Halliday (Chapter 2); Chris Collier (Chapter 3); Karan S. Bhatia (Chapter 4); Ahmed Fahmi (Chapter 5); Anthony Navoy (Chapter 6); Anne Coudrain (Chapter 7); Albert Rugumayo (Chapter 8); John Fenwich (Chapter 9); and Matthew Fry and Frank Farquharson (Chapter 10).

Peer review of the material prepared for the chapters was provided by the following experts: Robert Halliday (Chapter 2); Nicholas Kouwen (Chapter 3); Mauro Greppi (Chapter 4); Svein Harsten (Chapter 5); Giovanni Maria Zuppi (Chapter 6); Valerio Vendegna (Chapter 7); Filippo Thierry and Fabio Santamaria (Chapter 8); Maria-Monica Ghioca (Chapter 9); and Bruce Stewart (Chapter 10).

The following experts contributed to the revision of the chapters and sections (indicated in brackets) of Volume II of the Guide: Arni Snorrasson (Chapter 2); Paul Mosley (material taken from Chapter 2); Bruce Mitchell (Chapter 3); Tinus Basson (4.2); Suresh Chandra (4.3); P.B.S. Sarma (4.4); Valdemar Andrade (4.5); Denis Mosnier (4.5); Benno Droge (4.6); Carlos Tucci (4.7); Shankar B. Kulkarni (4.8); Carlos Meier (4.9); Kaz Adamowski (Chapter 5); Zbigniew W. Kundzewicz (Chapter 6); and Curt Barrett, Kosta Georgakakos, Ian Cluckie, Paul Pilon, Sergei Borsch and James Dent (Chapter 7). Contributions on the latest technological developments in remote-sensing were provided by Edwin Engman and Ahalam Shalaby in various chapters.

Peer review of the material prepared for Volume II was carried out by the following experts: Paul Pilon (Chapter 3); Richard Muller (4.2); Ponnuswarni Sooriyakumaran (4.3); Mario Fugazza (4.4); Valdemar Andrade and Denis Mosnier (4.5); Hussam Fahmy and Maha Tawfik (4.6); Jim Elliott (4.7); Christophe Ancey (4.8); Denis Hughes (4.9); Manuel Irigoyen and Ezio Todini (Chapter 5); Paolo Mignosa (Chapter 6); Ilmar Karro and Liljas Eric (Chapter 7). Giacomo Teruggi, John Bassier and Arthur Askew provided strong and essential support to the publication process through active coordination with the authors and performing the necessary technical editing. Above all, the publication would not have been possible without the active support of the WMO Secretariat staff.

The sixth edition of the Guide will be a living document and its Web version will be updated as and when there are significant developments in the practices in any particular field. As it is applied and used in practice, the Guide may be improved through comments and suggestions from the hydrological community. The Commission for Hydrology will endeavour to keep the Guide as up to date as possible by considering the feedback from its members.

(Bruce Stewart)
President, Commission for Hydrology

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Hydrology is the science that deals with the occurrence and distribution of the waters of the Earth in time and space, both above and below the land surface, including their chemical, biological and physical properties, and their interaction with the physical environment (WMO/UNESCO, 1992). It provides an understanding of various phases of water as it passes from the atmosphere to the Earth and returns to the atmosphere. As such, it forms the basis for water resources assessment and management and the solution of practical problems relating to floods and droughts, erosion and sediment transport and water pollution. Increasing stress on the available water resources in the search for improved economic well-being and concerns for the pollution of surface water and groundwater have highlighted the central role of hydrology in all water and environment initiatives.

To provide guidance in monitoring this vital resource, which is central to the development and well-being of humankind, the World Meteorological Organization (WMO) Commission for Hydrology, at its first session (Washington DC, 1961), recognized the urgent need for the preparation of a guide to the relevant operational practices. As a result, the first edition was published in 1965 as the *Guide to Hydrometeorological Practices*.

The second and third editions of the Guide were published in 1970 and 1974, respectively. The third edition was entitled *Guide to Hydrological Practices* in recognition of the broader scope of its contents. Subsequently, during its fifth session (Ottawa, 1976), the Commission approved the revision of and substantial additions to the Guide to produce a fourth edition, which was issued in two volumes. Volume I dealt with data acquisition and processing and Volume II with analysis, forecasting and other applications. Volumes I and II of the fourth edition were published in 1981 and 1983, respectively. With the evolution of technology and the Hydrology and Water Resources activities within WMO, the fifth edition of the Guide was published in 1994 as one consolidated volume. It was also published on a CD-ROM for easy outreach to a wider water management community beyond the traditional WMO constituency.

In 1999, the World Meteorological Congress adopted "Weather, Climate and Water" as the official subtitle of the Organization. The following year, the Commission for Hydrology, at its eleventh session in Abuja, Nigeria, recommended that the sixth edition of the Guide be published as a live document to be uploaded to the Internet and updated more frequently, as and when required.

1.2 SCOPE

The accepted principles of integrated water resources management dictate that, in order to achieve environmental sustainability and economic productivity, rivers must be managed at the basin level. Today, when water is perceived to be a matter of universal concern, various stakeholders, at the national as well as at international level, participate and play important roles in the process. Many institutions and agencies within a country are engaged in the collection of hydrological data and information. These data may be collected by various agencies using different measurement procedures. The resulting lack of homogeneity in the observations gives rise to a lack of confidence. It is imperative, therefore, that all these partners be made aware of the manner in which the hydrological data are collected, the limitations and the reliability of the data, and how they are to be managed by the responsible organizations in the basin. Transparency in data collection, storage and sharing is an essential element for cooperation among various users. A quality management framework for hydrometry and hydrological information is fundamental in using hydrological information from diverse sources.

The growing demand for freshwater resources has increasingly focused the attention of governments and civil society on the importance of cooperative management. Sharing the benefits of cooperation and even conflict prevention stem from a broad understanding of the principles and mechanisms through which these results can be achieved. Transboundary rivers have the potential to bring countries together both economically and politically or, conversely, they can cause economic and political tensions. The risk factor in decision-making in water resources management is a

function of hydrological variability. The risks can be mitigated through cooperative management of transboundary rivers. Cooperation in transboundary river management is fundamentally a political activity. Allocation of the resources or distribution of the benefits is essentially dependent on the knowledge of water availability and the related hydrological variability. A shared and accepted knowledge of the resources, their projected availability and the confidence in their accuracy greatly help in assessing the feasibility and fairness of alternative management and investment scenarios.

A lack of homogeneity in the data on the land phase of the hydrological cycle limits the scientific capacity to monitor changes relevant to climate and to determine the causes of variability and change in the hydrological regime. River discharge has a role in driving the climate system, as the freshwater flows into the oceans may influence thermohaline circulation. For easy and reliable use, the quality of such data and the procedures for its acquisition, storage and exchange should in general follow certain specified standards and protocols.

All of these factors increased the need for ensuring the quality of hydrological data. WMO, with a vision to provide expertise in international cooperation in weather, climate, hydrology and water resources, issues international guidance material and standards, and it is hoped that this Guide will form an important link in the quality management framework for hydrological practices. To meet such requirements, continuing efforts have been made to expand and improve the Guide, now in its sixth edition. It is expected that this Guide will be useful to agencies – not only to National Hydrological Services, but also to other stakeholders.

This Guide addresses all aspects of the land phase of the hydrological cycle, especially its phases upon and under the surface of the land. In conjunction with the manuals published by WMO, it provides detailed information on those areas that fall within the scope of the hydrology and water resources activities of the Organization designed to support National Hydrological Services and services with a similar mission.

The Guide forms part of an overall framework of recommended practices and procedures provided by *Technical Regulations* (WMO-No. 49) Volume III – Hydrology, as approved by WMO. Members are invited to implement these recommended practices and procedures in developing their Hydrological Services and activities.

1.3 CONTENTS OF THE GUIDE

It is difficult to set a clear dividing line between the science of hydrology and the practice of water resources planning and management. Nevertheless, for practical reasons, it was necessary to split the Guide into two volumes as shown in Figure II.1.1.

Volume I, entitled Hydrology – From Measurement to Hydrological Information, deals with networks, instruments, methods of observation and primary data processing and storage. It contains ten chapters, beginning with an introduction and an outline of the contents in Chapter 1.

Chapter 2, entitled Methods of observation, deals with the design and evaluation of hydrological networks and provides an overview of instruments and methods of observation for various hydrological elements that are described in detail in the subsequent chapters. Precipitation measurement in Chapter 3 is covered in all its aspects, ranging from the location of raingauges to the observation of precipitation by remote-sensing. The chapter covers liquid and solid precipitation, including their quality. Chapter 4, Evaporation, evapotranspiration and soil moisture, addresses both direct and indirect methods and also briefly reviews methods for evaporation reduction.

Chapter 5, Surface water quantity and sediment measurement, is pivotal and deals with measurement of flow in rivers and the capacity of lakes and reservoirs. It is also concerned with the measurement of sediment discharge. This subject matter is discussed in greater detail in the *Manual on Stream Gauging* (WMO-No. 519) and the *Manual on Operational Methods for the Measurement of Sediment Transport* (WMO-No. 686), to which the reader is invited to refer for more information.

Chapter 6, which is entitled Groundwater, is concerned with measurements from wells and the hydraulic properties of aquifers. It also looks in some detail at various remote-sensing techniques for groundwater observation.

The development of water resources is not only constrained by their quantity but also by their quality. Accordingly, Chapter 7, Water quality and aquatic ecosystems, addresses subjects ranging from sampling methods to remote-sensing. Chapter 8, Safety considerations in hydrometry, discusses topics ranging from the safety of personnel performing the measurements to safeguarding recording stations and the samples collected.

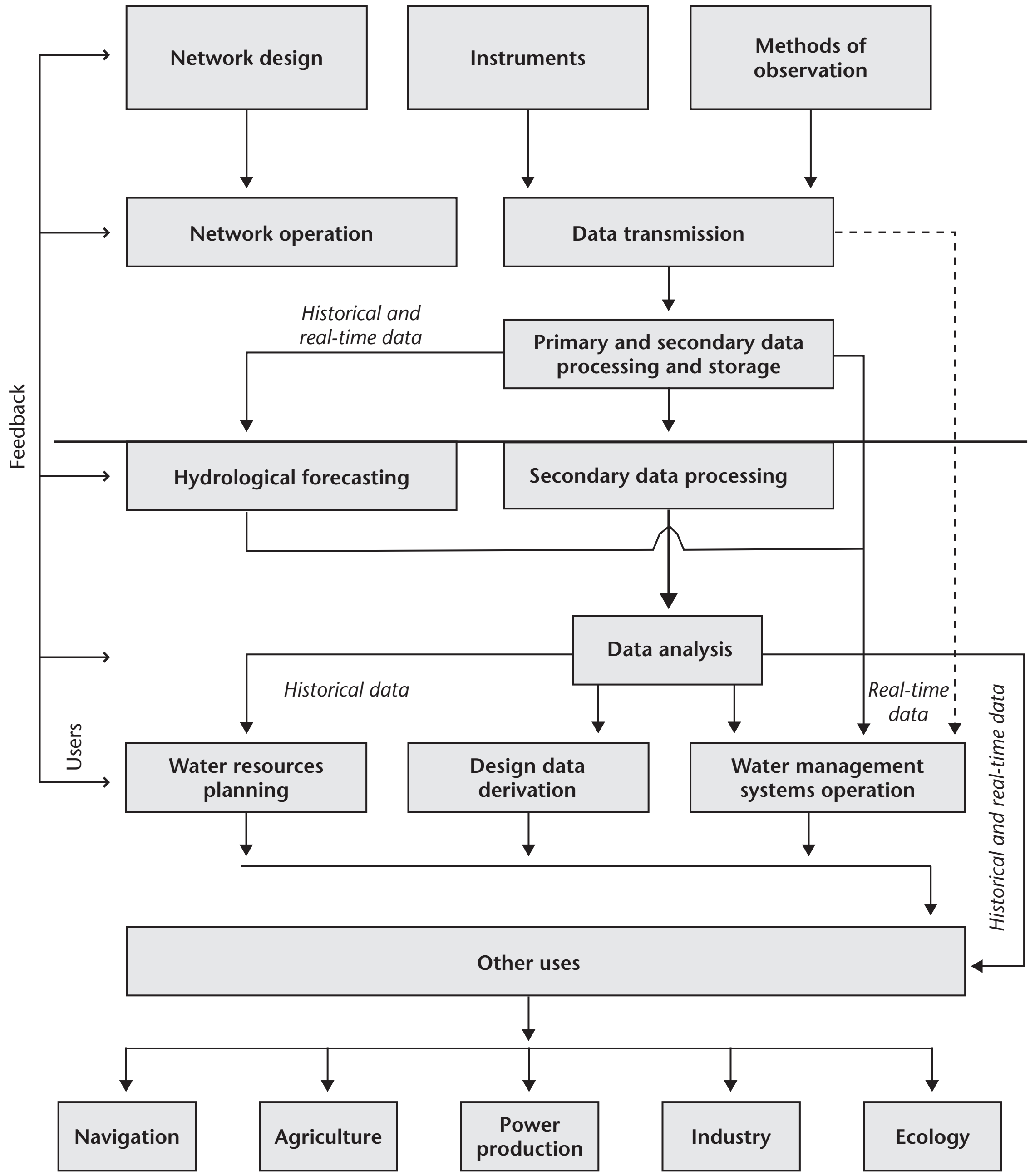


Figure II.1.1. Hydrological system

Lastly, Chapters 9 and 10, Data processing and quality control and Data storage, access and dissemination, respectively, include the dissemination of data for use by the wider water community.

Volume II deals with the application of the information referred to above in hydrological forecasting and the planning and design of various water projects. Accordingly, the volume is entitled Management of Water Resources and Application of Hydrological Practices. It consists of seven chapters beginning with an introduction and outline of the contents in Chapter 1.

Chapter 2 provides guidance on the management of hydrological services, including human resources aspects and financial and asset management. Chapter 3 introduces integrated water resources management and emphasizes the vital role of quality hydrological data in addressing various complex water management issues. Chapter 4 highlights the use of hydrological information in applications to water management, namely estimating reservoir

capacity and yield, flood management, irrigation and drainage, hydropower and energy-related projects, navigation and river training, urban water resources management, sediment transport and river channel morphology and environmental issues. Chapter 5 deals with extreme value analysis, and Chapters 6 and 7 address the modelling of hydrological systems and hydrological forecasting, respectively, as two of the key functions of Hydrological Services in water management.

While a measure of standardization is desirable and can be achieved with respect to instruments, methods of observation and publication practices, this is rarely the case with respect to hydrological analysis and applications. Therefore, the emphasis in Volume II is on presenting alternative approaches to the solution of selected problems, which have been demonstrated through experience to be both practical and effective. Rather than recommending one approach or technique in preference to another, attention is drawn to the principal features and advantages of each approach. The final choice will depend on a multitude of factors, including the relevant hydrological and climatic regimes, available data and information and the purposes to be served, and can only be made in the light of a full understanding of a specific situation. During the past few years, the increasing availability of microcomputers has permitted the introduction of more sophisticated analytical methods and techniques. Some of these have now been widely adopted and have therefore been introduced into this Guide.

The space limitations of this Guide restrict the amount of material that can be presented. For more detailed information on the subjects discussed, the reader should consult the following publications: for discharge measurement, the *Manual on Stream Gauging* (WMO-No. 519, Volumes I and II) and on sampling, the *GEMS/Water Operational Guide* (UNEP, 2005). The reader is also referred to international standards dealing with methods for liquid flow measurements in open channels prepared by member countries of the International Organization for Standardization (ISO). ISO has developed more than 50 standards for various types and methods of measurement. Valuable references can also be found in the proceedings of the international symposiums, seminars and workshops on hydrometry organized by the International Association of Hydrological Sciences (IAHS), WMO and the United Nations Educational, Scientific and Cultural Organization (UNESCO).

A full description of the theoretical base for the recommended practices and detailed discussion of their methods of application are beyond the scope of this Guide. For such details, the reader is referred to appropriate WMO manuals and technical reports, as well as to other textbooks, handbooks and technical manuals of national agencies. In particular, further detailed guidance on instruments and methods of observation is given in the *Guide to Meteorological Instruments and Methods of Observation* (WMO-No. 8) and the *Guide to Climatological Practices* (WMO-No. 100).

References and suggestions for further reading appear at the end of each chapter.

1.4 THE HYDROLOGICAL OPERATIONAL MULTIPURPOSE SYSTEM

In recent decades, hydrological science and technology have made substantial progress and significant contributions have been made by field hydrologists to the development and management of water resources. So as to facilitate the sharing of hydrological practices among the National Hydrological Services, a technology transfer system known as the Hydrological Operational Multipurpose System (HOMS) was developed by WMO and has been in operation since 1981. It offers a simple but effective means of disseminating information on a wide range of proven techniques for the use of hydrologists. HOMS transfers hydrological technology in the form of separate components. These components can take any form, such as a set of drawings for the construction of hydrological equipment, reports describing a wide variety of hydrological procedures and computer programs covering the processing and storage of hydrological data, as well as modelling and analysis of the processed data. To date, over 180 components have been made available, each operationally used by their originators, thus ensuring that every component serves its purpose and has been proved in practice. These descriptions appear in the *HOMS Reference Manual* (HRM) which is available online at http://www.wmo.int/pages/prog/hwrp/homs/homs_en.html in English, French, Russian and Spanish. The present Guide is further enriched through cross-references to the relevant HOMS components, which are included at the beginning of the relevant sections of this Guide.

References and further reading

United Nations Environment Programme Global Environment Monitoring System (GEMS)/Water Programme, 2005: *Global Environment Monitoring System (GEMS)/Water Operational Guide*. Fourth

edition, Inland Waters Directorate, Burlington, Ontario.

World Meteorological Organization, 1980: *Manual on Stream Gauging*. Volumes I and II, Operational Hydrology Report No. 13, WMO-No. 519, Geneva.

———, 1983: *Guide to Climatological Practices*. Second edition, WMO-No. 100, Geneva.

———, 1988: *Technical Regulations*. Volume III, Hydrology, WMO-No. 49, Geneva.

———, 1989: *Manual on Operational Methods for the Measurement of Sediment Transport*. Operational Hydrology Report No. 29, WMO-No. 686, Geneva.

———, 1994: *Guide to Hydrological Practices*. Fifth edition, WMO-No. 168, Geneva.

———, 1996: *Guide to Meteorological Instruments and Methods of Observation*. Sixth edition, WMO-No. 8, Geneva.

———, 2000: *Hydrological Operational Multipurpose System (HOMS) Reference Manual*. Second edition, Geneva.

———/United Nations Educational, Scientific and Cultural Organization, 1992: *International Glossary of Hydrology*. WMO-No. 385, Geneva.

CHAPTER 2

HYDROLOGICAL SERVICES

2.1 INTRODUCTION

Most Hydrological Services operate in the public sector and are therefore influenced by trends in governmental policy and practice. What is more, they work in a rapidly evolving environment characterized by the following factors:

(a) Heightened global commitment to sustainable management of natural resources and the environment, combined with efforts to improve the living conditions of the poor, who generally are the most dependent on natural resources;
(b) An expanding emphasis on the need for integrated water resources management, as pressure on the world's water and other natural resources creates a general awareness that resources must be developed and managed in a sustainable manner;
(c) A seemingly inexorable increase in the impact wrought by natural disasters, particularly floods and droughts. At the same time, risk management is becoming more widely adopted;
(d) Increased investments by Hydrological Services in capital assets and staff retraining, which is the cost of offering new or improved products;
(e) A mounting expectation that public services should be accountable not only to elected representatives, but also to the public at large. Public services should conduct their affairs with efficiency, effectiveness and economy. This expectation can culminate in the threat of litigation when the general public feels let down by government agencies;
(f) Keener competition for resources in the public sector, as governments seek to reduce taxation while meeting rising public expectations;
(g) The growing impact of globalization, which affects Hydrological Services both directly and indirectly;
(h) The impact of socio-economic trends on the day-to-day operations of Hydrological Services, such as the increasing involvement of women in professional activities and the ever-growing use of the Internet or the Web-based delivery of hydrological data and products.

Effective resource management requires accurate information as a basis for planning, implementing and monitoring resources. However, to be fully implemented, integrated water resources management demands a wide range of hydrological and related information, which may not be easily available. In order to obtain such information, a National Hydrological Service (NHS) requires proper institutional development to meet these new challenges and must develop appropriate capabilities and/or establish partnerships or strategic alliances with complementary agencies.

The general public constantly seeks higher-quality hydrological information and products. Such high-quality products and processes can provide a competitive edge for Hydrological Services that are required to compete in the marketplace. Maintenance of verifiable standards are also necessary and thus, as a general rule, a Hydrological Service should make quality management a focal point of its business, and its director should take ultimate responsibility for the quality of hydrological products.

Public agencies are continually required to do more with less, and often are obliged to recover some operating costs and seek commercially profitable projects in order to reduce the burden on the taxpayer.

As a result, the director of a Hydrological Service must constantly monitor changes in its environment so as to provide appropriate responses.

2.2 RESPONSIBILITIES AND FUNCTIONS OF HYDROLOGICAL SERVICES
[HOMS A00]

2.2.1 Nature of the products and services of a Hydrological Service

Hydrological data and information are, by and large, excludable public goods because the marginal cost of supplying the data to an additional client is practically zero, and yet access to the information can be controlled. A Hydrological Service may therefore choose to allow open access to some information, for example, making it accessible on the Internet, which places it irrevocably in the public domain, and to restrict access to other information, for example, only releasing it to selected clients, under strict contractual conditions regarding subsequent release. In this sense, services such as the

issuing of warnings to the public are pure public goods.

A National Hydrological Service can provide hydrological data and information at the national level in a cost-effective way. The implications of this are as follows:

(a) Products or public services, such as public warnings, can only be provided through public funding, because the Hydrological Service concerned cannot easily recover its costs from the beneficiaries;
(b) To obtain or sustain funding from government sources for the provision of both pure and excludable public goods, it is necessary to demonstrate their value or merit to society;
(c) Products or services that are excludable public goods can be provided on the basis of profitability or recovery of costs from the beneficiaries. Authority may be required for such operations. Proper accounting practices, financial transparency and fair charges will be expected;
(d) Boundaries between the management of pure public goods and excludable public goods may shift as a result of, for example, evolving technology, contractual arrangements and public information. A Hydrological Service may be able to influence such boundaries, if it is in the national interest;
(e) When lobbying for sources of funding, managers should review their products and services to ensure that they are in line with the mandate and pricing structure of the NHS.

2.2.2 Clients of hydrological products and services

Who are the clients of a Hydrological Service? In principle, for a National Hydrological Service, the ultimate client is the general public, represented by elected officials at the national, state/provincial and local levels. This includes the general public of the future, who will be the beneficiaries of hydrological work that is being done today.

Governmental policies and national development goals, and the information required to support them, influence an NHS fundamentally. For example, in many developing countries the beginning of the twenty-first century has seen a growing national emphasis on poverty alleviation. The management of a Hydrological Service should monitor governmental policies and analyse the implications of these policies for the individual Service. What products and services will the Hydrological Service need to provide in order to support national policies and goals? Do the Service's current products and services contribute to this? In other words, the management should ensure that the Service's products and services have the greatest possible value. This is best done objectively by means of cost-benefit, cost-effectiveness and poverty analyses, among others. The public's interest may be varied, so that a Hydrological Service may have a variety of clients, in addition to the traditional ones. A Hydrological Service may also offer private services to clients who are prepared to pay for them. The range of such clients will vary from country to country, depending on the nature of the national economy.

The management of a particular Hydrological Service must implement frequent surveys to identify potential clients. The management should continually monitor trends in demand for water, national and provincial policies and development goals, political manifestos, trends and events in various economic sectors, as well as international agreements and agreements with donor agencies and other development partners.

Client expectations of all businesses, including those in the public sector, are rising continually and businesses must continually seek to meet or exceed those expectations. Hydrological Services are no exception. To ensure the future of the Service, managers must encourage a client focus among their staff. The single most important client is the person to whom the director reports, for instance, the Minister for the Environment. The future of a Hydrological Service depends on how successfully the director markets the Service to that person and demonstrates how the Service can be useful to the client.

A marketing strategy should meet the following aims:

(a) Identify the current and potential clients of the Hydrological Service, and maintain and update a client database;
(b) Identify the products and services required by the clients which the Service can provide;
(c) Identify the most suitable mode or place of delivery of the product or service to the client, for example, use of the Internet to provide access to real-time data, fax warnings and conventional written reports with data annexes on CD-ROM;
(d) Determine a pricing policy for different products and services, and for different clients;
(e) Specify the types of people involved in delivering the product or service;
(f) Characterize the processes of product or service delivery according to the needs of the clients;

(g) Promote the Hydrological Services where potential clients can be clearly identified and contacted directly.

2.2.3 Managing relationships with clients

A crucial element of running any business is maintaining effective relationships with clients. Good communication with clients is essential in ensuring that they be made aware of the Hydrological Service's capabilities. Good public relations and honest feedback are necessary for client satisfaction. As members of the general public constitute the bulk of the clientele of a public-sector Hydrological Service, it is essential that the public be kept informed of activities and outputs, and be provided with opportunities for feedback. A Hydrological Service should have a high profile, that is, be visible, and ensure that the public be made aware of its work. World Water Day, held on 22 March of each year, provides such an opportunity.

2.2.4 Hydrological products and services

The basic products of a Hydrological Service are water-related data and information. Data and information are of value for decision-making. Hence, a Hydrological Service might be seen as providing increased confidence or reduced risk to its clients, as they make water-related decisions. In fact, one measure of the value of these data and information is their impact on the decisions that are made.

There is a continuum in the products that a Service might provide:

(a) Water-related data and observations obtained from an observing network. Hydrological database management systems provide basic statistics such as daily, monthly, seasonal and annual means or maxima, which are useful to clients;
(b) Water-related information, such as a comprehensive assessment of national water resources, the statistics of flood events or maps of spatial/temporal trends in groundwater quality;
(c) A monitoring service, designed to provide very specific data or information at a particular location for a particular client, for example, to indicate when the dissolved oxygen concentration downstream from an outfall falls below a specified minimum value;
(d) Knowledge and understanding of water-related phenomena and water resources;
(e) Advice on decision-making, where information is developed into recommendations for responses to certain conditions, for instance, advice on an appropriate response to a contaminant spill on a major river, or on how to respond to an evolving drought.

The management of a Hydrological Service should seek to develop added-value products and services, and to move out of the "data trap", in which the Service merely provides data from which other people extract value. Capacity-building, in terms of staff skills, information management technology, quality assurance and marketing will be necessary. Other changes may have to be made to institutional arrangements as well, for example, permitting a Service to retain the revenue that it generates. The products and services offered by a Hydrological Service have value and therefore are economic goods. Hydrological Service staff should learn to package their products to meet the needs of clients. They should also realize that client needs change with the evolving climatic and economic situation.

2.2.5 Functions and activities of a Hydrological Service

The functions of a Hydrological Service should reflect the products and services required by the client. The *Technical Regulations* (WMO-No. 49) set out the core functions of a Hydrological Service in Volume III – Hydrology, D.1.1, 8.3. These include the following activities: developing standards and quality assurance programmes; designing and operating observation networks; collecting, processing and preserving data; assessing user requirements for water-related data and information; and providing such data and information, for example, hydrological forecasts and water resources assessments.

Hydrologists today need a much broader view of hydrology, including ecological, biological and human-use aspects of the aquatic system. Accordingly, the activities of many Hydrological Services are becoming increasingly diverse as they deal with different types of data and information. Hydrological Services should continually monitor changing demands for water-related data, information and advice so that they can allocate resources appropriately. An early start is always desirable, because baseline measurements and trend information will be required for many purposes in the future.

Special national circumstances might require additional basic activities, such as monitoring river channel erosion and migration, or reservoir sedimentation.

The functions and activities of a Hydrological Service are not fixed in time, but change in response to the evolving needs and expectations of society and according to technological developments. The management of a Hydrological Service should continually monitor changes in its functional environment and assess their implications for the Service. For example, in recent years, the activities of some Hydrological Services have changed significantly in response to the following developments:

(a) Recognition of the hydrological significance of climate change, bringing about a new emphasis on drought monitoring, forecasting of extreme events and time-series analysis;
(b) The near-universal adoption of computer database management systems, leading to the publication of hydrological yearbooks and dissemination of hydrological data and products in electronic form;
(c) Cooperative agreements among the riparian States in transboundary river basins;
(d) Adoption of regional political agreements, with consequent changes and adaptations of standards, regulations and directives to which participating countries must abide, such as the European Union Water Framework Directive 2000/60/EC (EC, 2000), which has brought considerable change to Hydrological Services in both the European Union and potential member States.

In the past, the fundamental activity of a Hydrological Service was to design and operate a basic network of observing stations. This enabled a national assessment to be made of the country's water resources, thus providing a basic set of data to meet future needs for data at all locations and for a wide range of purposes. This also called for the technical capacity to provide estimates at locations for which there were no field data at all.

It is becoming difficult to sustain the concept of a basic national network in many countries. In some countries, promotion of integrated water resources management on a river basin basis, excellent though this concept may be, has led to monitoring efforts being focused on particular applications of data at the expense of national coverage. Therefore, it is important to demonstrate the benefits of a comprehensive, integrated Hydrological Service, in which broadly based data collection is more economical, both in the operation of monitoring networks and data management, as well as in the assessment of water resources.

A Hydrological Service may also engage in the provision of private services. Examples include the following:

(a) Compulsory monitoring of incoming water quality below a wastewater outfall for a factory;
(b) Monitoring of reservoir inflows and power station outflows for a hydropower company;
(c) Water-related data required for an environmental impact assessment for private use;
(d) Provision of information for a private irrigation company;
(e) Groundwater bore monitoring for a water supply authority.

These may require the establishment of special purpose networks or project networks (or individual stations) to meet specific client needs. The client would cover the cost and own the products, which could only be archived or disseminated upon the client's request. The management of a Hydrological Service should seek such opportunities. Special projects have many benefits for a Hydrological Service, including increased revenue, spreading of overhead costs across a wider range of clients, opportunity to develop new skills, heightened profile and support, and increased innovation. The data produced by these activities could also be integrated into information generated from the basic national network.

2.2.6 Evaluation of products and services, and quality management

The final stage in marketing is to obtain feedback from clients on the products and services. There are many ways of obtaining feedback. Perhaps the simplest is a friendly telephone call a few days after the product has been delivered, to enquire whether it has met expectations. More formally, clients may be asked to complete a simple questionnaire. Possibly the ultimate evaluation tool is to arrange a client satisfaction survey. Essentially, the aim of such a survey is to determine the original expectations of the client and measure the extent to which such expectations have been met. One way of ensuring customer satisfaction is to establish a quality management framework guaranteeing the development of products according to precise, replicable and agreed procedures and standards.

Defined standards are an essential basis for quality assurance of a Hydrological Service's products and services. Increasingly, clients require knowledge of the standards that are being achieved by the Service to satisfy their clients. In general, standards may be specified for the procedures that are used by the Service, and for the attributes of the products that

the Service produces. It is important to remember that standards are needed, not only for technical activities related to hydrometric data collection and provision of metadata, but also for all other activities undertaken within the Service, such as finances, staff performance and long-term planning. A review of International Organization for Standardization (ISO) standards for metadata from a WMO perspective can be found at http://www.wmo.int/pages/prog/www/WDM/reports/ET-IDM-2001.html.

Quality management should be carried out in a systematic way. In other words, a Hydrological Service should have a quality management system in place that assures clients that its products and services meet the standards of quality that have been defined for them. A Service may find that operating a well-documented quality management system can also be of great assistance should it become involved in legal proceedings related to its data and information products.

A comprehensive quality management system is often perceived as being expensive to implement. In practice, however, a quality management system should be no different from the data/product management procedures that the Hydrological Service uses to make measurements, convey them to the office, process and archive them, and transmit them to clients. Carrying out these procedures efficiently requires the following:

(a) Documented procedures for each step of the data and information flow;
(b) Defined standards for measurement and processing procedures, the measurements (data) themselves and derived products;
(c) Staff training and overview;
(d) Assigned responsibilities;
(e) Clearly documented data.

These elements of data management are also components of quality management. A comprehensive quality management system might include additional components such as the following:

(a) Verification that standard procedures are being followed, for example, by independent checks on flow-rating curves or field work;
(b) Validation that archived data meet defined standards, for example, by cross-comparison between neighbouring stations;
(c) Documented evidence that all aspects of the system are being consistently monitored, for example, a training record for each staff member.

Although the cost of quality, implicitly high quality, is commonly perceived to be high, the cost of poor quality may well be higher. A Service may discover that observations it had made over several years were worthless because of a hitherto unrecognized fault in an instrument, or that it must completely reprocess a flow record because a weir was incorrectly rated. Such remedial measures incur a much higher cost than would have been involved in initially checking the instrument or the rating.

2.2.7 Legal basis for operations and organizational arrangements

Almost all countries have Hydrological Services that have been explicitly established by some form of legal instrument or that carry out functions that are provided for or are enabled by legislation. However, in some cases, such legislation does not establish a specific agency or even identify a government agency that is required to collect hydrological information in the process of discharging its other responsibilities. In such cases, authority may come from an annual appropriation of funds, rather than the establishment of a law to include hydrological activities.

A great variety of legal or quasi-legal instruments giving varying degrees of authority is possible, for example, a national water policy, statute or law, water code, decree, order or inter-ministerial agreement, depending on the system of government. WMO (1994) provides a number of case studies that show the diverse arrangements that are possible. In many countries, water resources are now managed under the authority of a water law, a law establishing a water sector head office such as the National Water Resources Council, a law on environmental protection, a law on natural resources management, or similar statute. In these cases, the emphasis of the law is on aspects of resource management, such as allocation, resource pricing, or administration of permits. Hydrology may receive only passing reference, perhaps by being granted the authority to collect appropriate information.

There is a marked trend towards establishing river basin agencies that have comprehensive responsibilities for water management, including the provision of water-related information. Such agencies can now be found on every continent. In many cases, these river basins and their agencies are transnational, such as the Zambezi River Authority in Southern Africa. In some countries, there is complete coverage by river basin agencies, whereas in others only the principal rivers are covered. Management of water resources by river basin agencies or by subnational civil administrations introduces a need to harmonize standards,

coordinate data exchange, avoid duplication and assure national interests, for example. These tasks may be assigned to a national head office and carried out by its secretariat, which may be provided by the NHS or be completely separate. Inter-agency coordination and liaison are absolutely necessary in such cases.

The increasing complexity of decision-making in the water sector with a variety of stakeholders and actors, often with conflicting interests, necessitates a clear definition of the roles and responsibilities of each player. Furthering the main aim of integrated water resources management requires that data and information be available to all participants. Therefore, an appropriate legal framework and some form of legal instrument are desirable to provide a basis for a Hydrological Service's operations. In particular, such a legal framework may be needed to provide authority for activities or functions such as traversing private property as part of maintaining a monitoring station; charging fees for the delivery of products or services; requiring other organizations, including those of the private sector, to provide copies of their data for addition to the national archives; or transnational activities or liaison.

When laws related to water are being revised, managers should seek to participate in the drafting process. In particular, they should try to implement measures that have been successful in other countries and attempt to introduce these into their national legislation. Contacts with other organizations, such as WMO and its regional association working groups, will provide useful ideas. WMO publications also provide practical guidance (see WMO, 1994, 2001*a*).

The functions of a National Hydrological Service may be undertaken by a National Hydrometeorological Service, by one main sectoral Hydrological Service or by federal Hydrological Service overseeing various state or regional Hydrological Services.

In a survey of 67 countries carried out in 1991 (WMO, 2001*a*), four principal models were found for organizing Hydrological Services at the national level. Some 51 per cent of those surveyed had national hydrological or hydrometeorological agencies ; 1 per cent, regional (subnational) hydrological or hydrometeorological agencies; 42 per cent, both national and regional hydrological or hydrometeorological agencies; 6 per cent had neither national nor regional hydrological or hydrometeorological agencies.

The organizational arrangements of Hydrological Service are very diverse. Much depends on the legal system, governmental structure and stage of economic development, and successful examples suggest that effective operational hydrology can be conducted under a variety of circumstances. Although managers of a Hydrological Service may have limited influence on organizational arrangements at the national level, they should take every opportunity to participate in organizational restructuring. They should draw on the experience of Hydrological Service managers in other countries in order to propose changes that will help improve the performance of their Service.

At the level of an individual Hydrological Service, the organizational structure will largely depend on the Service's functions, products and activities. As these are always evolving, so too, should the structure evolve.

Managers of a Hydrological Service should draw on the experience of other Services when considering appropriate organizational structures. Extensive information on the relative merits of different organizational models, such as pyramidal structures or flat structures, is always useful.

In a country with several Hydrological Services, defined standards are of particular importance in ensuring comparability of hydrological data and products. A key role of the NHS or lead Hydrological Service is to establish national standards. The same could be said of an international river basin in which there are several NHSs. In this case, a key role for a river basin organization would be to establish standards for the whole basin and assist NHSs in achieving them.

The *Technical Regulations*, Volume III – Hydrology (WMO-No. 49) provides a set of long-established technical standards, as does the ISO Handbook 16 – Measurement of Liquid Flow in Open Channels (ISO, 1983).

The technical standards adopted by a Hydrological Service provide an objective basis for performance monitoring and appraisal. These standards should be incorporated into a Service's objectives.

2.2.8 Managing relationships with other institutions

Water is vital to many sectors of the economy, and many governmental as well as non-governmental organizations are likely to have an interest in water. Indeed, most countries have several organizations

engaged in different aspects of hydrology, with monitoring of surface water, groundwater and water quality commonly the responsibility of different agencies. Even if there is a designated NHS or a National Meteorological or Hydrological Service (NMHS), there are likely to be complex interrelationships among hydrological agencies. As integrated water resources management and river basin management principles become more widely adopted, relationships among water-related agencies will be consolidated.

Key areas of cooperation for Hydrological Services include the following:

(a) Data and information exchange among Hydrological Services under different parent organizations, and with the NHS if one exists;
(b) Cooperative arrangements that avoid duplication and facilitate sharing of technology, for example, through the joint operation of monitoring networks, shared facilities such as instrument calibration laboratories, joint purchasing arrangements for hydrological software or instrumentation, or joint field exercises for quality assurance;
(c) Data and information transfer to client organizations that require hydrological information for resource management or other purposes;
(d) Collaboration with national disaster management agencies and the NMS, so as to provide forecasts and warnings of extreme hydrological events;
(e) Joint projects in research and development with universities or research institutes, where the Service benefits from research and development, and the research establishment benefits from, among others, accessible data, field installations and opportunities for research students;
(f) Cooperation and liaison between Hydrological Services and the NMS for the exchange of water-related and climate data and shared technology for data management.

In most countries, cooperation in the water sector is deemed to be so important for the national interest that a central organization such as a National Water Resources Council is established. The position and authority of such bodies vary widely. In some cases they are largely advisory, with limited power. In others, they are chaired by or report directly to the Prime Minister, and have considerable authority. Hydrological Services always benefit from such arrangements.

Many riparian countries share river basins with others, and the downstream countries, such as Bangladesh, Cambodia, Egypt and the Gambia, are heavily dependent on upstream flows. Ideally, their NHSs should collaborate closely with those of the upstream countries so as to have the ability to forecast flows and issue warnings. River basin organizations, such as the Mekong River Commission (www.mrcmekong.org) and the International Commission for the Protection of the Rhine (http://www.iksr.org/), facilitate such relationships in some river basins, although this is not always so. Unquestionably, a key responsibility of the director of a Hydrological Service in a riparian State is to maintain close working relationships with those holding equivalent positions in the other countries concerned, either bilaterally, or in the framework of river basin agreements administered through multilateral river basin organizations.

A number of international organizations provide considerable assistance to national water resources agencies and Hydrological Services, and directors of Hydrological Services should be aware of the mandates and interests of those organizations.

2.2.9 **Data exchange**

Arrangements for data exchange are of considerable importance to Hydrological Services, including:

(a) The NHS, NMS and sectoral Hydrological Services in a single country;
(b) NHSs in a transboundary river basin;
(c) NHSs in neighbouring countries, with which water resources are not shared, but where data access would facilitate hydrological modelling or analysis;
(d) NHSs and international organizations concerned with global water resources assessment and international data archives;
(e) Hydrological Services working for national projects and to assist the private sector.

In the early 1990s, new technological developments and governmental policies posed a threat to the long-standing free and open exchange of meteorological data. Therefore, in 1995, the WMO Congress adopted Resolution 40 (Cg-XII) – WMO policy and practice for the exchange of meteorological and related data and products including guidelines on relationships in commercial meteorological activities – which explicitly excluded hydrological data. Four years later, in 1999, Congress adopted Resolution 25 (Cg-XIII) – Exchange of hydrological data and products (WMO, 2001*b*). This Resolution specifically relates to the international exchange of hydrological data and information products, but the basic principles are applicable at the national level. As discussed in 2.5.2, it is economically efficient to transfer or exchange data under a charging

regime in which only the transfer costs are levied, and this is essentially the principle expressed in Resolution 25 (Cg-XIII). A number of Hydrological Services have experimented with financial arrangements for data transfer in recent years, and the general consensus seems to be that the approach advocated by Resolution 25 (Cg-XIII) is preferred. In practice, the situation is more difficult in transboundary river basins, where issues of national sovereignty and national development outweigh all others. In these circumstances, NHSs can only insist that Resolution 25 (Cg-XIII) be followed.

Many Hydrological Services consider it useful to provide data to educational institutions and international scientific projects at no charge. On the other hand, if the data are to be used for consulting work, there is no reason why a Hydrological Service cannot require payment of a charge based on the cost of obtaining, verifying, storing and transferring the data concerned.

2.3 PLANNING AND STRATEGY [HOMS A00]

Perhaps a director's most important responsibility is to implement the Hydrological Service's planning and strategy development. To successfully respond to changing conditions and demands, a Service needs a director with vision and the ability to implement actions. Planning and strategy development imply change. Few people like change, especially when it is imposed on them, and the managers of a Hydrological Service need skill in managing change. In particular, the organizational culture of many Services may need to shift from one that has a technical focus to one that focuses first and foremost on clients.

Managers of a Hydrological Service need plans and strategies that ensure the Service allocates its resources to achieve its most important goals. Diverse plans of different duration should be formulated to match identifiable goals. A strategic plan will provide a view of the overall direction of the Service, for example, for a period of five years. In times of rapid change, it is difficult to look ahead even five years; therefore the plan would need to be updated regularly. An annual plan sets out the specific intentions and desired results to be achieved during a single year of operation; it is usually associated with a budget. A development plan focuses on the process of building a Service's capacity to carry on its business, and may consider a time period of 10 years or more for this purpose. In addition, there may be plans that focus on particular aspects of the Service's operations, such as a staff training plan.

A comprehensive plan is likely to include some or most of the following elements:
(a) Vision – how we want our world to be;
(b) Mission – the reason for which the Hydrological Service exists;
(c) Principles or values – the fundamental and unchanging beliefs that relate to the work of the Service;
(d) Review of achievements during the last planning period;
(e) Analysis of strengths, weaknesses, opportunities and threats (SWOT);
(f) Goals and desired outcomes – broad statements of what is to be achieved;
(g) Objectives and desired outputs – specific targets: measurable results and standards, together with a time frame;
(h) Actions – specific actions that will be used to achieve objectives and outputs;
(i) Financial budget;
(j) Performance criteria and indicators – measures that will be used to check progress.

A strategic or long-term plan would not specify actions and might include only an indicative budget. An annual plan, however, might briefly summarize many sections from an existing strategic plan and place more emphasis on defining the proposed actions and associated budget.

The above-mentioned list commences with the high-level vision and mission statement, continues with an appraisal of how the Service has performed, an honest evaluation of its condition (the strengths and weaknesses of the Service) and of its business environment (opportunities for new business and threats from competitors or adverse changes in the environment), and then moves on to the specification of actions and the means of measuring whether these have been successful. It is easy to obtain plans from other agencies to develop ideas on appropriate approaches and formats. The *WMO Strategic Plan* (WMO-No. 1028) should, for example, be available to a Hydrological Service director and may be obtained from the WMO Secretariat. Other services in the WMO group are an obvious source of guidance; for instance, the Australian Bureau of Meteorology (1995, 2005, 2006) has plans on a range of timescales, which might provide useful examples for other services.

A Hydrological Service that is a component of a parent organization may have a strictly defined

planning and budgeting format and process to which managers would adhere. However, where there is more freedom, managers should take planning seriously. At times, where resources are lacking and it appears as though the Service receives no recognition or encouragement, planning may seem to be a pointless exercise. However, it is perhaps under these conditions that planning is most necessary to set a positive course for the future and provide an impetus for change.

A plan is not solely an internal document, but is commonly used to promote the Service and as the basis for a performance agreement or contract between the director and the senior official to whom the director reports. In this case, the plan will be negotiated with the senior official, as well as the Service's staff.

Planning procedures are an essential component of management and are dealt with in many textbooks and all tertiary-level business management programmes. Hydrological Service managers should make planning a basic part of their business management studies.

Planning need not be technical or time-consuming, although techniques such as discounted cash flow analysis to select the most promising of several alternative courses of action can be used. It is, perhaps, most important to effectively involve stakeholders in the process, that is, not only senior management, but all the Service's staff, clients and potential collaborators. A mix of a top-down and bottom-up generation of ideas is desirable, facilitated by consultation with clients and other stakeholders. The director and senior management should set the overall direction of the Service, on the basis of their understanding of the wider business and political environment. Other staff may have a more hands-on perspective on strengths and weaknesses, and personal links with clients and collaborators. Commonly, individual departments will make proposals for components of the plan, which will be incorporated, modified or omitted according to the chosen selection procedure.

A useful starting point for appraising the present condition of a Hydrological Service is the *Water Resources Assessment: Handbook for Review of National Capabilities* (WMO/UNESCO, 1997).

Before a Hydrological Service's strategic, annual or other plan is implemented, managers must develop a clear link between the Service's plan and the responsibilities and duties of its staff. It is essential that managers focus the attention of their staff on the results that they are expected to achieve, and not simply on the tasks that they are to carry out.

An essential aspect of planning is appraisal of past performance. In many countries, government agencies are required to provide an annual report to the national assembly of elected representatives, and this provides the ultimate in performance appraisal. Even where they are not required to do so, directors of Hydrological Services should review at least annually their Services' activities, achievements and changing environment. The findings might be presented in different ways and degrees of detail for various audiences: for elected representatives and clients, brief, focusing on contributions to national life; for staff, detailed, highlighting technical and product/service matters; and for management and planning staff, comprehensive, including an analysis of deficiencies and adverse changes in the environment.

Appraisal of the performance of the entire Hydrological Service provides the basis for identifying its strengths and weaknesses, and for developing a plan that builds on these strengths and eliminates any weaknesses. Managers should appraise the Service's performance in terms of the performance criteria and indicators previously defined by the plan, and should consider how successfully the Service is achieving its vision and mission and governmental policies and goals. Feedback from public- and private-sector clients alike is an invaluable element of performance appraisal. A Service that delivers products that are technically first class but contribute little towards achieving governmental goals or meeting commercial clients' needs is unlikely to receive consistent support or funding for future planning periods.

2.4 HUMAN RESOURCES MANAGEMENT AND CAPACITY-BUILDING [HOMS A00, Y00]

2.4.1 Management

Most organizations consider their most important resource to be their staff. This is true and managers of successful Hydrological Services know this well. As the role and functions of a Hydrological Service evolve, the staffing requirements and management style of the Service may need to change as well. Hence, a Service that is modernizing or developing value-added products, for instance, is likely to require more staff skilled

in information technology. Such staff will perform their tasks differently from staff with traditional field skills and will require different levels and styles of supervision.

The long-term success and health of a Hydrological Service rests in the hands of the director and managers. To discharge their responsibilities effectively, they require skills in a number of areas. The director should ensure that the entire management team has the following assets:
(a) Diplomatic and administrative skills to function successfully in the public service environment or as a State-owned company;
(b) The ability to monitor and understand the business environment and translate it into planning the Service's programmes;
(c) Skills in all areas of business management – human resources, finances, capital assets, product quality, information technology – as appropriate to the Service;
(d) Leadership skills and motivation;
(e) Marketing and communication skills that are needed to develop effective relationships with clients, the public and elected representatives, investors/donor agencies and the "owner";
(f) Technical and scientific knowledge required to ensure that the Service has the technology it needs;
(g) The ability to represent the Service and the national interest at an international level.

The director should place as much emphasis on management training as on technical capacity-building.

If indeed staff are the most important resource, managers should select staff with great care. They should appoint or reassign staff to meet the demands of the Service's strategic and annual plans so that work groups have the human resources needed to achieve their assigned objectives. A director should take staff succession seriously, that is, identification and preparation of junior staff to advance to more responsible levels as senior staff retire. A combination of experience and training will be needed to suitably prepare such staff.

A contract between the Hydrological Service and an employee is an essential basis for effective and fair human resources management. Legal requirements with regard to employment contracts vary from country to country, and managers of Hydrological Services should be familiar with the employment-related legislation under which they operate. For a Hydrological Service that is a parastatal or state body, the form of contractual arrangements for employees is normally specified by national civil service regulations.

The Director of a Hydrological Service that lacks closely specified arrangements for employee contracts should seriously consider developing them. The main benefit of a contractual agreement, for both employer and employee, is that the relationship is specific and transparent, allowing any shortcomings on either side to be addressed in an objective manner.

A job description for every staff member is an essential management tool. It provides both a clear statement of what the Service expects of the individual and a basis for setting personal objectives, implementing performance appraisals and identifying training and personal development opportunities.

Job descriptions and objectives provide the basis for the appraisal of staff performance, which is on a par with planning and strategy development in terms of importance to the managers and staff of a Hydrological Service. In many organizations, performance appraisal is linked to preparation of staff development plans for individual staff members, for work groups – for example, a work group that is expected to take on new responsibilities – or for the entire Service. Staff development plans will be used for future performance appraisals, in part to ensure that proposals have been implemented and to evaluate their success as tools for enhancing performance.

When human resources management tools such as job descriptions, setting of objectives and performance appraisal are introduced, staff members can be resistant and sceptical. However, a manager will find that they will be more likely to cooperate if tools to enhance their prospects are used sensibly, constructively and persistently over a period of one or two years. It cannot be overemphasized that management tools must be used with understanding; if not, they are likely to be of little value, and even counter-productive. This implies that the director and managers of a Hydrological Service should ensure that their own performance meets the Service's needs.

2.4.2 Training and continuing education

Training and continuing education are of critical importance to both management and staff, the collective goal being to enable staff members make the greatest possible contribution to achieving the Service's mission. Training and education should

be managed in a structured way, possibly by preparing a training plan for the Service or for individual staff members. It should correspond to training needs analyses that are part of the performance appraisal process. These analyses may also be carried out independently, for example, when managers are considering new procedures, products or services, organizational restructuring or some other response to the changing business environment, and when there is a need to match current competence with that of the past.

2.5 FINANCIAL AND ASSET MANAGEMENT [HOMS A00]

Financial management has become a basic aspect of a director's work, as governments worldwide impose more stringent financial disciplines. Normally, financial management procedures are defined by a Hydrological Service's parent organization, and the director and/or selected management staff receive appropriate training in these procedures. Nevertheless, a director should make every effort to develop a much more sophisticated grasp of financial management than the basic minimum.

Accounting procedures in the public sector are generally prescribed by government and a Hydrological Service that is part of a government department or State-owned enterprise will have to follow these scrupulously. This is to ensure transparency and accountability, that is, to make sure that the Service's financial accounts are clear and comprehensible, resources are spent for the designated purpose, responsibilities for financial transactions can be identified and funds are not siphoned off through corruption – unfortunately a fact of life in developed as well as developing countries.

2.5.1 Sources of revenue

A major concern of managers of a Hydrological Service – indeed, of any organization – is the source of income or revenue required to maintain the Service's operations and assets. In most countries, the government has been and will continue to be the predominant source.

The recent trend worldwide is for governments to require or enable public sector agencies to find sources of commercial revenue in addition to allocations from the national budget. Some Services have made significant progress in identifying non-governmental clients or willing clients within the public sector. Value-added products and services are the most profitable. A Service should focus its energy on seeking new sources of revenue only in areas that are consistent with its primary mandate and where a good, that is, profitable, business case can be made.

Commercial work requires a legal mandate, and the managers of Hydrological Services that engage in commercial activities should be familiar with national laws and regulations relating to commerce.

In most countries, there are relatively few non-governmental clients for value-added products that are potential sources of considerable commercial revenue. This is especially the case in developing countries where the pressure on a Hydrological Service to find supplementary sources of revenue is also likely to be greatest. Most of the products and services of a Hydrological Service, and the databases and other assets that are needed to provide those products and services, are, however, public goods for which the government is the logical purchaser. A Hydrological Service may nevertheless be required to recover some of the costs of its public services and products.

Economic theory indicates that the appropriate approach to cost recovery is to charge for the direct and associated overhead costs of providing the product or service, including the administrative cost of recovering costs, as well as depreciation of the assets used. Where the product or service uses a hydrological database or other asset provided at public expense, it is economically inefficient to attempt to charge part of the cost of providing that asset. Potential clients strongly object to such charges and refuse to use the service at all. This results in underutilization of the public asset, the use of inferior alternatives such as guesswork and hence economic inefficiency. The experience of Hydrological Services that have attempted to charge for data generally confirms this. Increasingly, it is regarded as preferable to provide unrestricted public access to data via the Internet at no charge. This reduces the cost of meeting data requests and can help enhance the Service's reputation.

As a means of imposing financial discipline and achieving maximum transparency, a government may choose to administer funds in ways other than making an allocation in the national budget. These include:

(a) Providing funds through a non-governmental organization such as a National Research Council, which allocates funds on a competitive basis and/or in terms of defined national needs for information;

(b) Establishing the Hydrological Service as a State-owned enterprise and administering public funds on the basis of a contract for defined outputs and services. In the extreme, the contract could be awarded on a competitive basis, potentially to another provider;
(c) Introducing a government contract between the appropriate Minister and/or the Minister for Finance and the director of the Hydrological Service, to provide defined outputs and services.

The director of a Hydrological Service is unlikely to have much influence on such a decision, which will reflect overall government policy. However, the director should seek guidance from other directors in similar circumstances, either in other organizations in the same country or Hydrological Services in other countries, and attempt to negotiate contractual arrangements that provide the most favourable conditions for future work.

Lastly, it is worth recalling that a way of increasing effective revenue is to reduce costs, for example, by moving from paper-based to Internet-based dissemination of information. Of course, the Service should ensure that the quality of the product or service does not suffer, but is preferably enhanced from the user's perspective.

2.5.2 Budgeting and monitoring financial performance

Budgeting should be an integral part of annual planning. As the Hydrological Service defines its proposed programme of objectives and activities, it will need to define the associated costs and, through an iterative process, revise the proposed programme so that its cost is consistent with likely revenue. Just as it is desirable for operational staff to be involved in annual planning, so too should they be involved in setting the budget. They will, after all, have to work with and within it.

As a rule, Hydrological Services that are parastatal or state bodies, budgeting procedures and timetables are strictly defined. The annual planning process must therefore be timed accordingly. The Hydrological Service is likely to be required to submit its budget in a defined format to its parent organization, in terms of specified line items in a chart of accounts. The managers of the Service should ensure that the internal process of preparing a budget provides an end result that can readily be converted into the required format, but may prefer to use a format that is more appropriate to the Service's business or simpler to use.

The completed budget should be a key component of the annual plan and a means of monitoring performance against the plan.

2.5.3 Asset management

In simple terms, the purpose of asset management is to ensure that the value of the organization's assets is maintained, and therefore that the organization continues to be a going concern that has the resources to do business. Therefore, it is of considerable importance to all managers and staff of a Hydrological Service. Asset management basically involves acquisition, replacement, maintenance, protection and disposal of assets.

2.5.4 Database security

A Hydrological Service's single most important asset is its database. Means of protecting this asset will depend on the data storage media that are used, but there is no doubt that the director of a Service must ensure that it is protected. In a number of countries, data rescue projects have been necessary to gather together all data – original records, usually on paper – place them in a secure location and convert them into an electronic format that is more manageable on a long-term basis. Such projects, admirable though they may be, should become necessary only under circumstances completely beyond the control of the Hydrological Service. There can be few excuses for a director who allows his Service's basic asset to be degraded.

Paper media, for example, observers' notebooks, recorder charts and machine-punched tapes, are invaluable because they usually provide the original record that must be consulted if questions arise about data validity, or if data reprocessing is required for some reason. They should be stored in such a way that they will be subject to the smallest possible degree of damage by insects, water, rot, sunlight, fire, earthquake or simple loss. Original records should be the responsibility of a single office; if such an arrangement is not possible, however, the same person in each office in which the records are stored should be responsible for them. Whatever the case, the location of original documents should be carefully tracked, for example, if they are released for reprocessing. If a Hydrological Service does not have the facilities or expertise to permanently archive its paper media, the national archive, museum or library may be able to assist.

As paper media are subject to deterioration, copies should be made. Commonly, microfilm or

microfiche copies are made, but the obsolescence of this technology presents difficulties for the future. Electronic storage of scanned images is now an economical alternative, using CD-ROMs or other even higher density media. In this case, technological obsolescence is perhaps an even greater concern than microfilm; thus the Hydrological Service will need a procedure for regular migration of electronic archives onto successive generations of storage media.

The secure long-term storage of original and processed records in electronic form, for example, incoming telemetered data, or an entire computer database, requires procedures that are not so much sophisticated as disciplined. It is essential to make regular, frequent backups of data, following rigorously defined procedures so that data are not lost before they reach the archive, and to make liberal comments on archived data so that subsequent users can understand any changes that have been made.

References and further reading

Australian Bureau of Meteorology, 1995: *Long-term Plan 1995–2010*. Commonwealth of Australia, Bureau of Meteorology, Melbourne.

———, 2005: *Strategic Plan 2005–2010*. Commonwealth of Australia, Bureau of Meteorology, Melbourne (http://www.bom.gov.au/info/leaflets/strategic-plan-2005-10.pdf)

———, 2006: *Operational Plan 2005–06*. Commonwealth of Australia, Bureau of Meteorology, Melbourne.

European Commission, 2000: *Directive 2000/60/EC of the European Parliament and of the Council of 23 October 2000 establishing a framework for Community action in the field of water policy*. Water Framework Directive, European Union, Brussels (http://www.wmo.ch/pages/prog/www/ois/Operational_Information/coolC14Sky/VolumeD/Amendments/AdditionalDataProducts/02_Resolution%2040.pdf).

International Organization for Standardization, 1983: *Measurement of Liquid Flow in Open Channels*. ISO Standards Handbook 16, Geneva.

World Meteorological Organization, 1994: *The Legal Basis and Role of Hydrological Services* (M.P. Mosley). (WMO/TD-No. 602), Geneva.

———, 1995: *Abridged Final Report with Resolutions of the Twelfth World Meteorological Congress* (WMO-No. 827), Geneva.

———, 1999: *Abridged Final Report with Resolutions of the Thirteenth World Meteorological Congress* (WMO-No. 902), Geneva.

———, 2001*a*: *The Role and Operation of National Hydrological Services* (P. Mosley), Technical Reports in Hydrology and Water Resources No. 72 (WMO/TD-No. 1056), Geneva.

———, 2001*b*: *Exchange of Hydrological Data and Products* (P. Mosley), Technical Reports in Hydrology and Water Resources No. 74 (WMO/TD-No. 1097), Geneva.

———, 2006: *Technical Regulations*. Volume III – Hydrology (WMO-No. 49), Geneva.

———, 2007*a*: *Basic Documents*. No. 1 (WMO-No.15), Geneva.

———, 2007*b*: *WMO Strategic Plan* (WMO-No. 1028), Geneva.

———/United Nations Educational, Scientific and Cultural Organization, 1997: *Water Resources Assessment: Handbook for Review of National Capabilities*. Geneva and Paris.

CHAPTER 3

INTEGRATED WATER RESOURCES MANAGEMENT

3.1 INTRODUCTION

3.1.1 Sustainable water development

Since the 1970s, there has been a growing awareness that natural resources are limited and that future development must come to terms with this fact. The concept of sustainability has on the whole gained wide acceptance, although its meaning may vary, depending on the person. Sustainable development as defined by the International Union for Conservation of Nature (IUCN), the United Nations Environment Programme (UNEP) and the World Wildlife Fund (WWF) is adopted in this Guide: "improving the quality of human life while living within the carrying capacity of supporting ecosystems". (IUCN/UNEP/WWF, 1991).

Is there any way to measure the sustainability of development? If account can be taken of natural variability and trends in water resources availability, it is arguable that the effects of development will be reflected in changes in the resource base. The so-called ecological footprint is a tool that is used to measure the amount of land and water areas required to produce the resources it consumes and absorb its wastes (see, for example, http://www.footprintnetwork.org/). It has been estimated that today's global population has an ecological footprint that is 20 per cent larger than the biocapacity of the Earth. Monitoring of the quantity and quality of water in natural systems – streams, lakes, underground, snow and ice – thus becomes a prerequisite for tracking the extent to which development can be sustained.

The building of adequate databases through the monitoring of hydrological systems is a fundamental prerequisite of water resources assessment and management. This chapter reviews the adequacy of current monitoring networks and techniques in the light of a changing resource base and evolving water management philosophies related to sustainable development.

3.1.2 The changing nature of the resource

3.1.2.1 Natural changes

The hydrological system, driven by meteorological conditions, is constantly changing. Over long periods of time – decades to millenniums – variations in the receipt of energy from the Sun, acting through the atmospheric system, cause important changes in hydrological regimes. For example, changes in the distribution and extent of ice masses and vegetation cover usually reflect hydrological changes.

Recently, there has been increasing awareness that interactions between the air and the sea have extremely important effects on climate. El Niño Southern Oscillation events, for example, with teleconnections over wide areas, may have far-reaching hydrological ramifications, which are particularly relevant when associated with droughts and floods. Longer- term atmospheric phenomena such as the Pacific Decadal Oscillation and their teleconnections may also affect hydrological systems.

Natural events of a completely different type, such as major volcanic eruptions with massive emissions of dust and gases into the atmosphere, can also impact the hydrological system significantly.

3.1.2.2 Human-induced changes

Human activities increasingly affect hydrological systems. Some of the more important activities are listed below:

(a) The construction of dams and diversions has a major impact on flow regimes and sediment transport in many of the world's rivers, as well as on ecological systems in donor and recipient basins;
(b) Changes in land use often produce major effects on hydrological regimes as follows:
   (i) Deforestation often leading to more pronounced flood peaks and increased soil erosion;
   (ii) Draining of wetlands, often bringing about changes in the runoff regime;
   (iii) Frontier road and railway construction causing erosion, changes in human settlement and land-use change;
   (iv) Farming practices, resulting in varying infiltration rates and groundwater recharge;
   (v) Urbanization, prompting characteristically flashy runoff;
(c) The quality of water in many places has been adversely affected by industrial and municipal

waste and agricultural practices such as the use of fertilizers and pesticides;

(d) The emission of greenhouse gases leading to climate change and related changes to hydrological systems. According to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC, 2007), the duration, location and frequency of extreme weather and climate events are likely to change, and would result in mostly adverse impacts on biophysical systems;

(e) Long-range transport of air pollutants can cause ecological damage far from the emission sites.

Monitoring systems should take into account these many changes in order to better understand the hydrological system, predict water availability and manage resources effectively. In particular, anthropogenic climate change driven by significant increases in atmospheric greenhouse gases over the past few centuries and the resulting effects of climate change on hydrological systems pose enormous challenges for water managers. Given the uncertainty in regional climate scenarios and the likelihood that younger water resources managers will witness events not previously recorded in human history, there is an even greater need for quality assured hydrological datasets and robust physically based models than ever before.

3.1.3 Changing attitudes to management

There have been significant socio-economic changes in many parts of the world. Rapid population growth, particularly in developing countries and in burgeoning urban centres, combined with industrialization and rising living standards, have increased the demand for water. Pollution in many regions has reduced the quantities of safe drinking water. Groundwater levels have declined in many regions. Growing demand, outstripping supply, will become more common. Thus, more efficient and effective water management is imperative.

The past few decades have witnessed dramatic changes in water management. There have been two important underlying themes. First, there is a growing awareness that water is a fundamental element in the natural environment. The presence and movement of water through all biological systems is the basis of life. Water, land and biological systems must be viewed as interlinked, and monitoring of the various components of the ecosystem should be harmonized. Secondly, water is absolutely essential to all forms of economic activity, for example, agriculture and food

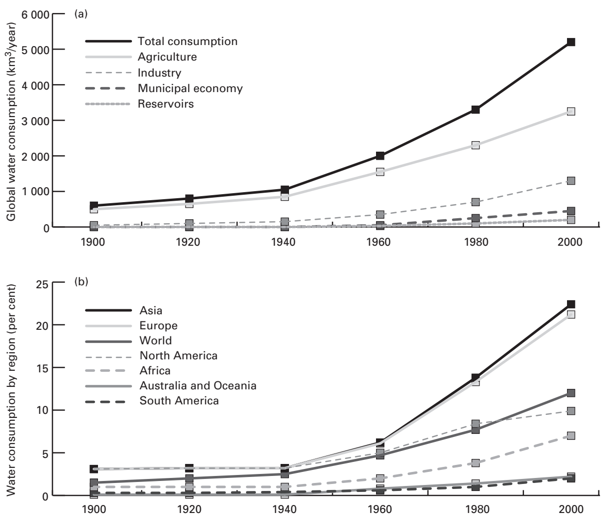


Figure II.3.1. Trends in world water consumption by (a) activity and (b) region

production, for much of industrial production and for the generation of energy. Water is also a critical factor in human health. Too much water, in the form of floods, or too little, such as drought, can lead to human and environmental disasters.

Figure II.3.1(a) shows trends in world water consumption from 1900 to 2000. Globally, consumption during that period increased tenfold; by the year 2000, almost half of the available water supplies was in use. Agriculture, particularly irrigation, remained the primary consumer despite a continuing decline in water use from 90.5 per cent in 1900 to 62.6 per cent in 2000. During the same period, industry's share of water consumption rose from 6.4 per cent to 24.7 per cent; that of cities with the same growth rate climbed from 2.8 per cent in 1900 to 8.5 per cent in 2000 (United Nations, 1997).

How has water consumption compared with the available water resources in each of the world's major regions during the twentieth century? Figure II.3.1(b) answers this question in terms of percentages calculated on the basis of theoretical resources, that is, the amount of water flow in rivers. According to these calculations, Europe and Asia clearly consumed much greater shares of their water resources than North America, Africa and, particularly, Australia and Oceania, and South America . It is also clear that Europe and Asia had the highest growth in consumption, except for South America, where the increase was offset by plentiful reserves of water.

Growing awareness of the pervasive nature of water, in addition to its importance in the natural

environment and in human activity, has led to the recognition that a holistic approach to its management is necessary. Development of the resource for human use may have a detrimental environmental impact while, conversely, changes in the natural resource base may limit or otherwise affect human activities. These changes have led to the holisitic approach known as integrated water resources management.

3.1.3.1 Watershed management

There is general recognition that the natural management unit is the river basin. It makes sense to manage the water resources within a river basin and in a coordinated manner, as the water is often used several times as it moves from the headwaters to the river mouth. It also makes sense to manage all natural resources – vegetation, soils and the like – within the basin unit. Water demands for human activities should also be managed within the basin in an integrated manner.

Unfortunately, political boundaries do not normally coincide with basin boundaries. Rivers often cross international frontiers and traverse states or provinces within countries. Globally, about half of all the land surface falls within international basins and more than 200 significant basins are international in character.

3.1.3.2 Management fragmentation

It is common that several agencies or institutions within a State have authority over different aspects of water resources management. Departments or ministries of environment, agriculture, energy, industry, and health often have conflicting mandates.

All too often, the monitoring networks within a State are also fragmented politically and institutionally. Even within single agencies, the responsibilities for water quantity and water quality monitoring often are not coordinated. Vague institutional responsibilities and mandates within countries, and conflicting demands on water use between countries (within international basins) or inter-state disputes within federal States, pose real problems for the establishment and maintenance of effective monitoring networks.

It is against this complex background of rapidly changing water management philosophies, political and socio-economic realities and the resource base itself that several actions must be taken. These include the design and operation of monitoring systems, the storage and dissemination of data, followed by the use of those data as the basis on which sound decisions can be taken to plan, design and operate water projects and issue warnings and forecasts of hydrologically significant events.

3.2 INTEGRATED WATER RESOURCES MANAGEMENT [HOMS A00]

The term integrated may be defined as having all parts combined into one harmonious whole, coordinating diverse elements.

Integrated water resources management can be interpreted at three different levels. First, it involves the systematic consideration of various dimensions of water: surface and groundwater, and quantity and quality. The key is that water represents an ecological system, containing interrelated parts. Each part can influence, and be influenced by, other parts, and therefore needs to be planned for and managed with regard to those interrelationships. At this level, attention normally is given to how to integrate considerations related to water security and water quality.

At the second level, managers recognize that while water is an ecological system, it also interacts with other resource systems, ranging from terrestrial to other environmental systems. This second level is broader than the first, and turns attention to matters such as flood-plain management, drought mitigation, erosion control, irrigation, drainage, non-point sources of pollution, protection of wetlands and fish or wildlife habitat and recreational use. At this level, integration is needed because many water problems are triggered by land use or other development decisions involving major implications for aquatic systems.

The third level is broader still, and directs the manager toward interrelationships among the economy, society and the environment – of which water is but one component. Here, the concern is the extent to which water can facilitate or hinder economic development, reduce poverty, enhance health and well-being and protect heritage.

All three levels highlight the fact that planners and managers deal with a mix of systems, which often involves hierarchical relationships. As a result, a key feature of integrated water resources management is the application of a systems or ecosystem

approach. Another key feature is the need to be focused and results oriented, as there is always a danger of defining systems or issues so broadly that they become impractical from a management perspective.

To quote the Inter-American Development Bank (1998), integrated water resources management involves decision-making on development and management of water resources for various uses, taking into account the needs and desires of different users and stakeholders.

In sum, the keys to effective integrated water resources management are a systems perspective, a focused and results-based approach, and partnerships and stakeholders. In the present chapter, attention is given to the rationale behind these aspects, how they have been applied in practice, what general lessons have been learned and what cautions should be borne in mind.

3.3 RATIONALE FOR INTEGRATED WATER RESOURCES MANAGEMENT

3.3.1 Water quantity and quality

The responsibility for managing the quantity, or supply, and quality of water is often assigned to separate agencies. This can be attributed to historical administrative reasons that are unrelated to the subject at hand, but also to the rationale that such a division nurtures efficiency because it enables professionals to focus on a specific aspect of water management. This practice has generally resulted in two groups or cultures of water professionals – managers of clean water and managers of dirty water – who operate separately.

A major disadvantage of this separation of authority for water quantity and quality is that the causes of, and therefore, solutions to, quantity and quality problems are frequently interdependent. For example, if flow in a river system drops because of natural variability, there may not be enough water to meet water-use needs or sufficient capacity to assimilate wastes deposited in the river. As a result, dams and reservoirs may be constructed in order to enhance storage to meet user needs and to provide augmented flow in the dry season in order to meet water quality standards. To achieve the optimum design of such dams and reservoirs, water quantity and quality needs should either be considered jointly or integrated in management practices.

3.3.2 Surface water and groundwater

In many regions of the world, groundwater is the major source of the flow in surface streams during the dry season. In addition, certain land-based activities, such as those causing leakage from underground storage tanks, can lead to pollution of aquifers. Other land-based activities, for instance, withdrawal, which is implemented to meet urban or agricultural needs that exceed rates of recharge, can also bring about the depletion of groundwater reserves.

Given the interconnections identified above, in order to achieve effective management of aquatic systems, it is necessary to study and manage surface water and groundwater as connected systems, particularly to ensure secure water supplies of acceptable quality. An integrated approach encourages – indeed, requires – the joint management of surface water and groundwater systems.

3.3.3 Upstream and downstream considerations

Decisions or action taken in the upstream part of a river basin or catchment have implications for downstream areas. For example, point and non-point pollution entering a river in the upper part of a basin may produce negative health or other impacts on downstream users, whether human or other species. Conversely, if officials in downstream urban areas determine that they can reduce their vulnerability to flooding by building storage dams and reservoirs in the upper part of their basin, then the upstream residents may suffer. This happens through inundation of urban and agricultural land caused by reservoir backwater, leading to loss of housing and livelihood for some farmers, and sometimes damage to, or loss of, heritage or areas such as burial grounds or historical sites.

The interconnections between areas of a river basin or catchment are often cited as a compelling reason for using the basin or catchment as the spatial unit for integrated water resources management. Such a rationale is logical. However, it must be understood that the relevant basin or catchment for surface water may not coincide with the spatial extent of an aquifer. It should never be assumed that surface and groundwater systems have the same spatial extent. The possibility of such a disconnection of the spatial boundaries of surface and groundwater systems poses a challenge to water managers, for which there is no obvious answer. Another challenge arises when interbasin transfers occur, requiring a perspective extending beyond the

upstream and downstream needs in one basin, so as to consider the interconnections between two or more river basins.

Another challenge for defining the spatial boundaries of a management system based on ecosystem characteristics is the presence of various administrative and political boundaries. Rivers, and sometimes lakes, have been used to delineate boundaries between municipalities, provinces, states and countries and are shared by several countries or subnational administrative units. As a result, management of such rivers and lakes requires the involvement and collaboration of various partners. The most flagrant example is the Danube river, whose basin is shared by 19 countries. Ensuring that upstream and downstream interests and concerns are addressed in situations involving different countries poses a significant challenge for implementing an integrated approach.

3.3.4 Water, land and other resource systems

Many water problems originate on land. To achieve flood damage reduction, for example, it is generally not sufficient to manage or control the variability of water levels in rivers and lakes through dam, dykes and levees. Land-use activity related to urban development and agriculture can result in the removal or shrinking of wetlands, forest systems and grasslands, which in turn exacerbates erosion and flooding problems. Indeed, it is claimed that much flood damage along the Ganges river in India and along the Indus river in Pakistan can be attributed to the removal of forests in the Himalayas. Furthermore, initiatives to enhance water quality must often start with attention to activity associated with other resource systems. Thus, the use of pesticides, herbicides and fertilizers to improve agricultural productivity is often a major contributor to non-point sources of pollution, requiring attention to land-based activities to tackle the pollution of water systems.

Another challenge is the long-range transport of airborne pollutants. Even if an integrated approach is taken by key managers within the basin, they usually do not have the authority to deal with sources of pollution originating outside the basin, at times hundreds of kilometres away.

3.3.5 Environment, the economy and society

Historically, water management has been dominated in developed and developing nations by three professions: engineering, agriculture and public health. As a result, engineers began focusing on structural solutions for issues ranging from water security – whether for urban, industrial or agricultural use – to water quality and flood damage. In addition, health professionals started turning their attention to the treatment and disposal of sewage and other wastes detrimental to health.

The domination of water management by engineering and health professionals led to an emphasis on technical and economic perspectives. During the 1960s, there was a growing awareness that environmental aspects should receive greater attention, followed by the recognition that social or cultural issues also required special consideration. Such recognition led to a gradual acceptance of the desirability of teams – multidisciplinary, at least, or interdisciplinary, at best – to gather and integrate a range of professional and disciplinary views in developing management approaches. While each discipline represented in a multidisciplinary team yields discipline-specific results and leaves the integration of the various contributions to a third party, building an effective team requires overcoming many obstacles and challenges. This is necessary to develop and apply new knowledge, with team members working together as equal stakeholders to address a common challenge. However, such teams are essential if the intent is for strategies to address environmental, economic and social aspects in an integrated manner.

3.3.6 Vertical and horizontal fragmentation: systems and silos

Notwithstanding the compelling reasons for using integrated water resources management, there are pragmatic reasons for public agencies being structured to focus on one or a subset of resource systems. Hence, it is common to find separate departments or ministries of agriculture, forestry, wildlife and natural resources. The separation of functions into different agencies is known as horizontal fragmentation when, for a given level of government – national, state or local – responsibility for a particular resource is assigned to various agencies. Such arrangements require a range of technical expertise represented on a team that can concentrate on issues and opportunities related to that resource and, where appropriate, develop working relationships with users of the resource. Along with such organizational structures, inter-departmental committees or task forces may be used to coordinate different interests, mandates and perspectives.

Without coordination and collaboration, there is a real danger of losing effectiveness and efficiency. For example, as a ministry of agriculture carries out its mandate to increase lower-cost food production, it may seek to drain wetlands to put more land into farming, or it may encourage the use of fertilizers or other chemicals to boost crop production. In contrast, a ministry of natural resources may introduce programmes aimed at protecting or expanding wetlands in order to enhance wildlife habitat and capacity, and delay runoff during storms, thereby reducing downstream flooding. Such programmes can also serve to discourage the use of agrochemicals in order to reduce pollution of waterways used by fish, birds and other species. The activities of the aforementioned ministries might not result in a net change in the amount or type of wetlands in a jurisdiction, while expending significant funds to drain wetlands in some areas and enlarge them in others.

As mentioned above, horizontal fragmentation refers to the division of responsibility within one level or layer of government. Vertical fragmentation occurs when agencies at different levels of government – national, state or local – share an interest in or responsibility for a resource, such as water. For example, a State agency might design, build and operate a dam and reservoir, one purpose of which is to provide water for nearby communities. At the same time, a local government agency might be responsible for distributing the water from a reservoir to households, industry and farm irrigation systems. When this occurs, there is a real need for mechanisms or processes to coordinate mandates and activities among the levels of government. Vertical and horizontal fragmentation can create obstacles to integrated water resources management; therefore, integration is essential.

One way to overcome such fragmentation has been to do away with specific resource-oriented agencies and create more broadly based ones, such as ministries of the environment or of sustainable development. This was done by the Canadian Federal Government, which eliminated its Inlands Water Directorate and re-distributed the professional staff among various divisions within Environment Canada. The aim was to ensure that water was considered along with other resource issues in order to achieve sustainable development. The theory was logical. However, it soon became apparent that other federal staff or clients had great difficulty finding the water specialists with whom to raise their concerns. Furthermore, it became clear that most farmers or other water users thought of themselves as having a water or waste problem, rather than a sustainable development problem.

The concepts of vertical and horizontal fragmentation illustrate that water and other resource managers face their greatest difficulties in handling so-called edge or boundary problems – those between the mandates and responsibilities of two or more agencies, and thus must be dealt with in a shared or partnership manner. Agencies usually do a very competent job dealing with problems or tasks that are clearly within their mandates and authority. In contrast, those with edge or boundary characteristics offer serious challenges, and thus call for an integrated approach, despite the practical administrative problems to be overcome.

3.3.7 Collaboration, coordination and coherence

Which criteria should be used to evaluate the success of a given management approach? It is common for the following criteria to be applied: effectiveness, in terms of achieving desired outputs or outcomes; efficiency, with regard to producing the desired effects without wasting time and energy; and equity, ensuring a fair distribution of benefits and costs of the desired results. The discussion in the previous subsections indicates that many aspects can hinder effectiveness, efficiency and equity.

Integrated water resources management is a tool that can help managers meet such criteria. The following factors are essential to achieve integration: collaboration, or the act of working together; coordination, that is, harmonious adjustments or working together, or, arranging in proper order; and relationships and coherence, that is, logical connection or consistency, harmonious connection of the parts of a whole.

Integration is a means to an end, not an end in itself. As a result, the use of integration in water management should be preceded by a shared vision about a desired future condition or state. Without such direction, it is difficult to determine which parts need to made into a whole, who should be working together to establish a proper order and relationships and what logical connections need to be made.

The rationale for integration as one tool to help in achieving a vision is to allow a desired future condition to be achieved effectively, efficiently and equitably. Integration is usually advocated because of its potential contribution to all three criteria. It

contributes to effectiveness by helping ensure that different needs and opportunities are considered and incorporated into plans and activities; to efficiency by helping ensure that actions of one agency or organization do not undo the actions of another agency; and to equity, by forcing consideration of different values and interests of various stakeholders.

3.4 EVOLUTION OF INTEGRATED WATER RESOURCES MANAGEMENT

Integrated water resources management is not a new concept. It has existed in one form or another for well over half a century in the voices and writings of eminent water experts such as Gilbert White. In 1977, the United Nations Water Conference in Mar del Plata adopted a resolution promoting the concept. Later, integrated water resources management was highlighted as a guiding principle contained in the 1992 Dublin Statement on Water and Sustainable Development (ICWE, 1992). More recently, the Global Water Partnership programme has been based on this concept. (Tortajada, 2003). In various countries and regions, initiatives have been taken to manage water using integrated water resources management as a basis, even if the term was not being used. This section reviews experience with selected approaches to integrated water resources management.

3.4.1 United States of America: Ohio conservancy districts, Tennessee Valley Authority

In 1933, the Tennessee Valley Authority was established so that initiatives related to hydropower development, navigation and flood control in the Tennessee river basin could be pursued in a coordinated and integrated manner. Without the Authority, different agencies responsible for power supply, navigation and flood control would most likely operate independently, and thus miss an opportunity to design and operate activities to complement one another. In addition, the Tennessee Valley Authority became involved in other initiatives, such as rural planning, housing, health care, libraries and recreation, because no agencies provided such services or facilities.

3.4.2 Canada: conservation authorities

Legislation was passed in 1946 in Ontario to create conservation authorities, catchment-based organizations formed through a partnership of municipalities with the provincial government (Richardson, 1974; Mitchell and Shrubsole, 1992). The trigger was the realization that individual municipalities did not have the resources or authority to take initiatives such as the construction and operation of upstream dams and reservoirs for flood damage protection, which would benefit an individual municipality, as well as other downstream communities. In 2005, there were 36 conservation authorities in Ontario, covering areas in which over 90 percent of the population lived.

Conservation authorities were founded on the following principles, which have had enduring value:

(a) The best management unit was the watershed: Many of the economic staples of the province, such as agriculture and timber, depended on water and terrestrial resources, highlighting the need for an integrated approach;
(b) Local initiative was essential: A conservation authority would only be established when two or more municipalities in a watershed agreed to collaborate with each other and the provincial government;
(c) Provincial-municipal partnership was key: Although the provincial government would not impose a conservation authority, it would participate as a partner. However, this feature also meant that areas with few people or a modest tax base would not be able to form a conservation authority, because there would be no local capacity to raise the required funds;
(d) A comprehensive perspective was required: Many land-based problems were caused by too much or too little water, and water-based problems often were influenced by land-based activities. Thus, a comprehensive approach was promoted, meaning that water and associated land-based resources would be considered together;
(e) Coordination and cooperation were important: Any new conservation authority was required to create links with provincial and municipal agencies responsible for other natural resources, the environment and planning.

3.4.3 United States and Canada: the Great Lakes

The Great Lakes basin, shared by the United States and Canada, covers a surface area up to the outlet of Lake Ontario of 765 990 km^2, 521 830 km^2 of which is land and 244 160 km^2, water. It contains about 20 per cent of the world's surface freshwater supply, and is home to over 40 million people – 14 per cent of the total United States population

and 30 per cent of the total Canadian population. Its governance involves two national, eight state, two provincial and hundreds of municipal governments.

The two national governments signed the Boundary Waters Treaty in 1909, and through that treaty created the International Joint Commission. It has six commissioners, three from the United States and three from Canada. It is a permanent bi-national body and forum for international cooperation and conflict resolution regarding air pollution, water quality, regulation of water levels and water flows between Canada and the United States along their common border. The Commission has quasi-judicial, investigative and surveillance roles and two operational arms: the Great Lakes Water Quality Board and the Great Lakes Science Advisory Board. The Commission drafted two important agreements, the 1972 and 1978 Great Lakes Water Quality Agreements, with amendments enacted in 1987. The 1978 agreement was the catalyst for the application of an ecosystem approach.

3.4.4 Australia: total catchment management

Burton (1986) may be credited with the development of total catchment management in Australia. In 1947 the State Government of New South Wales created the Conservation Ministry to coordinate the management of the water, soil and forest resources in the state. In 1950, legislation was passed to create the Hunter Valley Conservation Trust, with responsibility for the coordinated management of water and land resources in that valley, inland from Newcastle on the coast of New South Wales. At a state-wide level, the State Premier in 1984 approved the creation of the Inter-Departmental Committee on Total Catchment Management and subsequently announced that a total catchment management plan would be developed for each of the major river valleys in New South Wales.

3.4.5 New Zealand: Resource Management Act

New Zealand's experience with integrated water resources management goes back to the 1940s (Memon, 2000). Later, starting in the 1960s, catchment control plans for soil conservation and river control were initiated, and these were followed in the 1970s with basin-wide resource inventories and informal water allocation plans.

The Resource Management Act of 1991 was major milestone, distinguished by its provision of "a statutory basis for a relatively integrated approach to environmental planning" (Memon, 2000). Furthermore, the Act replaced a large number of separate and sometimes inconsistent and overlapping acts related to the use of land, water, air and geothermal resources. Under this law, duties are divided among three levels. The central government focuses on policy and monitoring. At the subnational level, water and other resource and environmental management tasks are undertaken within a two-tier system involving directly elected multiple-purpose regional councils and territorial local authorities: city and district councils. The 12 regional councils are set up according to major river basin catchments.

3.4.6 South Africa

The Water Act of 1956 was introduced to achieve a fair allocation of water between competing agricultural and industrial needs. A key aspect of the legislation was giving water rights to riparian property owners, leaving them free to retain water through dams and other means. By the mid 1990s, however, it was recognized that the 1956 statute had some serious limitations. First, water quality concerns were not being systematically incorporated into management decisions which normally emphasized water quantity allocation. One result was growing organic pollution and eutrophication. Second, water requirements for the environment were not being adequately recognized. Third, at least in many rural areas, access to water was viewed as inequitable. Lastly, several analyses during the 1990s had noted the need for a more integrated approach to water management (Department of Water Affairs and Forestry and Water Resources Commission, 1996; Lazarus, 1997; Gorgens and others, 1998). The outcome was the White Paper on a National Water Policy for South Africa, which had been started in 1995 through consultations which extended over two years (Department of Water Affairs and Forestry, 1997).

South Africa introduced the new Water Services Act in 1997 and the National Water Act in 1998 to alter the way in which water was managed. A key aspect of the new approach was the incorporation of integrated water resources management. The National Water Act recognized that water was a scarce and unevenly distributed national resource and also a resource that belonged to all people, not just riparian landowners. Sustainability and equity were

identified as fundamental principles, and meeting the basic needs of present and future generations was a key objective, along with protecting the environment and meeting international obligations for shared water resources. Social and economic development was also to be promoted through the allocation and use of water.

The National Water Act emphasizes decentralization. Key new institutions include catchment management agencies, through which responsibility for water management is delegated to the regional or catchment level and which involve local communities. Each catchment management agency is responsible for a catchment management strategy, and through it the agency has powers to manage, monitor, conserve and protect water resources; make rules to regulate water use; require the establishment of management systems; and temporarily control, limit or prohibit the use of water during periods of water shortage. Water user associations also are established under the legislation, with the main purpose of helping and coordinating individual water users.

Based on the experience in South Afric[illegible] (1999) concluded that the successful u[illegible] grated water resources managemen[illegible] recognition that integrated water resourc[illegible] ment has the following characteristics:

(a) It is a team business. A team require[illegible] understanding and individual and t[illegible] and skills, rules and regulations fo[illegible] and proper organization. This also requires coaching, coordination, policies, integrated strategies and planning;
(b) It is about winning and achieving goals. This calls for commitment and passion for the game, individual and team motivation, team spirit, and mutual trust and respect for the game, the team and supporters;
(c) It is about superior strategies. This requires understanding of the real business, involvement of the right players and champions, addressing value systems, tactical organization, entrepreneurship, boundarylessness, innovation and the creation of a winning culture;
(d) It is about champions, people with vision, initiative and passion and outstanding leadership;
(e) It is an exercise in public administration and political science. There must be support for the programme or it will fail.

The above points are relevant beyond South Africa, and deserve attention when designing, establishing or implementing integrated water resources management strategies.

3.5 PERSPECTIVES ON INTEGRATED WATER RESOURCES MANAGEMENT

3.5.1 Dublin Conference: Earth Summit, 1992

Prior to the United Nations Conference on Environment and Development, commonly known as the Earth Summit, in Rio de Janeiro in June 1992, the International Conference on Water and the Environment was held in January 1992 in Dublin, Ireland. It was convened by WMO on behalf of all nations with an interest in freshwater. This was the most all-embracing event focused on global water issues since the United Nations Water Conference in Mar del Plata in Argentina in March 1977. The purpose of the Dublin Conference was to identify priority issues related to freshwater, and to recommend actions to address them (ICWE, 1992). The ideas and proposals from Dublin were taken to the Earth Summit, and many of the recommendations were subsequently included in Agenda 21, the strategy for sustainable development in the twenty-first century (Young and others, 1994).

[illegible]ublin Statement on Water and Sustainable [illegible]pment, the main output from the confer[illegible]merged from deliberations by more than 500 [illegible] from 114 countries, 28 United Nations agen[illegible]d organizations and 58 non-governmental [illegible]ergovernmental organizations. The pream[illegible]he Dublin Statement asserts that concerted action is needed to reverse trends of over-consumption, pollution, and rising threats from both floods and droughts. Action needs to come from local, national and international levels, and four principles guide future initiatives. The first principle has been interpreted as a call for integrated water management:

> Fresh water is a finite and vulnerable resource, essential to sustain life, development and the environment. Since water sustains life, effective management of water resources demands a holistic approach, linking social and economic development with protection of natural ecosystems. Effective management links land and water uses across the whole of a catchment area or groundwater aquifer.

The other principles emphasized the following needs:

(a) A participatory approach, involving users, planners and policymakers at all levels, with decisions to be taken at the lowest appropriate level;

(b) An enhanced role for women in the provision, management and safeguarding of water;
(c) The recognition that water has economic value in all its competing uses and thus should also be considered an economic good. Managing water as an economic good was viewed as an important way to achieve efficient and equitable use, and to encourage conservation and protection of water resources. In addition, all human beings have the basic right to access to clean water and sanitation at an affordable price.

The first principle of the Dublin Statement, a call for a holistic approach which, since the Earth Summit has usually been referred to an integrated approach, is the principle which has received the most attention. It emphasized that water problems cannot be treated in isolation, and indeed should be considered in relation to land-based and land-use planning issues. This was not a revolutionary principle, as the Organisation for Economic Cooperation and Development (1989) had previously published guidelines for integration relative to water management. Subsequently, researchers such as MacKenzie (1996) urged adoption of an ecosystem [holistic] approach, noting that it "can be seen as both comprehensive (in scope) and integrated (in content)".

Agenda 21 was the key outcome of the Earth Summit (United Nations, 1992). Chapter 18 of the Agenda deals with freshwater resources and provides a compelling rationale for integrated water resources management:

> ...the holistic management of freshwater as a finite and vulnerable resource, and the integration of sectoral water plans and programmes within the framework of national economic and social policy, are of paramount importance for action in the 1990s and beyond. The fragmentation of responsibilities for water resources development among sectoral agencies is proving, however, to be an even greater impediment to promoting integrated water management than had been anticipated.

In this assessment, Agenda 21 highlights the challenge of edge or boundary problems, noted earlier, as well as the significance of vertical and horizontal fragmentation.

3.5.2 World Water Council and the World Water Fora

The World Water Council was established in 1996 to provide an open platform for the discussion of water issues. Detailed information on the Council may be found on its Website, http://www.worldwatercouncil.org/.

The Council is an initiative of water specialists, the academic community and international agencies. It regularly convenes World Water fora to discuss water issues, develop proposals for action and highlight the importance of water. The First World Water Forum was held in Marrakech, Morocco, in March 1997, the second, in The Hague, Netherlands, in March 2000, the third, in Tokyo, Japan, in March 2003 and the fourth, in Mexico City, Mexico, in March 2006. The Fifth World Water Forum is planned to be held in Istanbul, Turkey, in March 2009.

The World Water Fora have been consistent in endorsing integrated water resources management. For example, the Ministerial Declaration of The Hague on Water Security in the 21st Century stated as follows:

> The actions advocated here are based on integrated water resources management, that includes the planning of management of water resources, both conventional and non-conventional, and land. This takes account of social, economic and environmental factors and integrates surface water, groundwater and the ecosystems through which they flow. It recognizes the importance of water quality issues.

The Ministerial Declaration emerging from the Third World Water Forum in Japan also endorsed integrated water resources management:

> Whilst efforts being taken so far on water resources development and management should be continued and strengthened, we recognize that good governance, capacity-building and financing are of the utmost importance to succeed in our efforts. In this context, we will promote integrated water resources management.

3.5.3 Global Water Partnership

The Global Water Partnership was set up in 1996, the same year as the World Water Council. Detailed information can be found at http://www.worldwatercouncil.org/.

The Partnership is an international network with support from a number of countries and international funding agencies. Its mandate is to support integrated approaches to sustainable water

management consistent with the Dublin and Rio principles by encouraging stakeholders at all levels to work together in more effective, efficient and collaborative ways. Its primary function is to encourage the exchange of information and, quite explicitly, to promote integrated water resources management. As an international network, it is open to all bodies involved in water management – governments of developed and developing countries, United Nations agencies, multilateral banks, professional associations, research institutes, the private sector and non-governmental organizations.

The Global Water Partnership network includes a secretariat in Stockholm and nine technical advisory committees for each the following regions: Southern Africa, West Africa, the Mediterranean, Central and Eastern Europe, Central America, South America, South Asia, South-East Asia and China.

3.5.4 World Summit on Sustainable Development, Johannesburg, 2002

The World Summit on Sustainable Development was held ten years after the Earth Summit in Rio de Janeiro. A plan of implementation was prepared to build on and extend the actions proposed in Agenda 21. Section IV of the Plan addresses matters related to protecting and managing the natural resource base of economic and social development, and the first topic covered was an integrated approach to their management.

With respect to an integrated approach, the Plan stipulated as follows:

> Human activities are having an increasing impact on the integrity of ecosystems that provide essential resources and services for human well-being and economic activities. Managing the natural resources base in a sustainable and integrated manner is essential for sustainable development. In this regard, to reverse the current trend in natural resource degradation as soon as possible, it is necessary to implement strategies which should include targets adopted at the national and, where appropriate, regional levels to protect ecosystems and to achieve integrated management of land, water and living resources, while strengthening regional, national and local capacities.

With reference to freshwater, the Plan stated that the objective should be to develop integrated water resources management and efficiency plans by 2005, with support to developing countries, through actions at all levels to "develop and implement national/regional strategies, plans and programmes with regard to integrated river basin, watershed and groundwater management".

3.6 ELEMENTS OF BEST PRACTICE FOR INTEGRATED WATER RESOURCES MANAGEMENT

3.6.1 Alternative interpretations: comprehensive versus integrated approaches

At the beginning of this chapter, the term integrated was defined as having all parts combined into a harmonious whole or coordinating diverse elements. This definition has led to integrated water resources management being characterized as a systems, ecosystem, holistic or comprehensive approach. However, emphasis on having all parts combined into a harmonious whole has also provided integrated water resources management with its greatest challenge.

At the strategic planning level, it is appropriate to interpret integrated water resources management as a comprehensive approach that seeks to identify and consider the broadest number of variables that are significant for the coordinated management of water and associated land and environmental resources. However, if such an interpretation is continued at the operational level, experience has shown that this contributes to inordinately lengthy periods of time needed for planning, and also results in plans which are usually insufficiently focused to be of value to managers.

Given the above challenges, a comprehensive approach should be used at the strategic planning level to ensure that the widest possible perspective is maintained, in order to avoid overlooking any key external or internal variable or relationship. However, at the operational level, more focus is needed. In that regard, an integrated approach, while maintaining interest in systems, variables and their interrelationships, is more selective and focused, concentrating on the subset of variables and relationship judged to be the most important and amenable to being influenced by management actions. If such a distinction is made between comprehensive and integrative interpretations of a systems, ecosystem or holistic approach, it should be possible to complete planning exercises in a

more reasonable length of time, identify the most important priorities for action, and thereby meet the needs of managers and users (Mitchell, 1990).

3.6.2 Vision for a desirable future

Integrated water resources management is a means to an end, not an end to itself. As a result, before its implemention, or as an initial step in such a process, it is important to have a well-established vision or direction about a desired future condition for an area or catchment. Integrated water resources management will be one instrument to assist in its achievement.

A vision articulates the destination towards which a group or society agrees to aim. The vision represents a future which in significant ways is better or more desirable than the present. Without such direction, it is difficult to determine which parts need to be brought together into a whole, and who should be working together to arrange a proper order and establish relationships.

Developing a shared vision can be a major challenge, since at any given time a range of values, interests and needs will exist among different stakeholder groups in a river basin or catchment. However, if there is no sense of direction, or clearly defined ends, integrated water resources management will not be able to create one. Thus, planners and managers must understand that without a vision it is unlikely that integrated water resources management will be an effective tool. Even worse, it may be discredited because it did not deliver a vision, something it was never intended to do.

When thinking about a vision for the future, it is helpful to distinguish among what is most probable, desirable and feasible. Planners and managers often focus first on identifying most probable futures, and insight on this is very valuable. However, too often, they stop there, or then move directly to considering what would be feasible futures, in light of what is deemed as most probable. An important point to remember is that the most probable future may not be the most desirable future; that is precisely why planners and managers seek to create a vision – to determine the desired future condition.

3.6.3 Spatial scale: watershed, subwatershed, tributary and site

It is important to make a distinction among different situations when applying integrated water resources management. The need to adjust the amount of detail included as spatial scale changes is especially significant. In a report focusing on lessons learned and best practices related to watershed management, three Ontario conservation authorities (2002) shared some interesting insights.

In Ontario, watershed planning, equivalent to integrated water resources management, is conducted on four different scales, "with the level of detail increasing as the size of planning area decreases". In that context, the most logical and efficient way to conduct integrated water resources management is to start with a catchment or river basin plan, then develop sub-catchment or subwatershed plans on a priority basis, and subsequently follow those with tributary plans, and finally with environmental site plans, as appropriate. Key lessons indicate that what is done at each stage provides direction and information for the next lower level and also helps avoid or minimize the potential for duplication.

However, financial constraints often result in subbasin or sub-watershed plans being prepared first, and integrated later into an overall basin or catchment plan for integrated water resources management. In a similar way, tributary plans may be completed before the sub-catchment plans. The three Canadian conservation authorities distinguish among the four levels of integrated water resources management in the following ways:

(a) Basin or catchment plans: Such plans cover large areas. These plans include goals, objectives and targets for the entire basin and document both environmental resources and environmental problems. They also provide catchment-wide policy and direction for protecting surface and groundwater, natural features, fisheries, open space systems, terrestrial and aquatic habitats, and other important features;
(b) Sub-basin or sub-catchment plans: These plans involve a smaller area compared with the basin or catchment level plan. On this spatial scale, enhanced detail is provided to allow local environmental issues to be addressed. Goals, objectives and targets for management of the sub-catchment are identified. Sub-basin or subcatchment plans dealing with integrated water resources management are custom designed to reflect local conditions and concerns. Recommendations may be included subsequently in official plans, secondary plans, growth management plans or other municipal planning instruments;
(c) Tributary plans: Plans on this scale are usually prepared to guide proposals for significant land

use changes, such as proposals for sub-divisions, large-scale water taking, gravel extraction, intensive agriculture and industrial estates. These are prepared for a portion of a sub-catchment and generally cover an area ranging from 2 to 10 km^2. Ideally, the boundaries of a tributary plan should align with the drainage basin of a tributary, but this is not always possible in practice. Recommendations emerging from tributary plans generally appear in secondary land-use plans, official land-use plan amendments, conditions for draft plan approval or for site plan approval;

(d) Environmental site plans: Such plans are usually developed to meet conditions set out in a draft plan. They provide details on proposed environmental and stormwater measures, and are usually submitted in parallel with plans for grading, erosion or sediment control and site servicing. Recommendations from environmental site plans normally appear in engineering design drawings for draft plans for a subdivision or industrial estate.

The four scales or levels identified above deserve attention from all planners and managers involved in integrated water resources management. By being aware that various levels of detail are appropriate on different spatial scales, planners and managers can increase the likelihood that issues and problems will be addressed at a suitable level of detail, overlap or duplication of work will be avoided, the time needed to complete integrated water resources management plans will be reduced and capacity for implementation will be boosted. If all of these are accomplished, the credibility and value of integrated water resources management will be enhanced.

3.6.4 **Partnerships and alliances**

Integrated water resources management was designed to ensure a holistic or ecosystem approach, and to facilitate the coordination of initiatives by different stakeholders. With regard to the latter, a strong motivation is required to break down what is often referred to as the silo effect, or the tendency of agencies to take decisions with regard only to their own mandates and authority, without reference to those of other organizations. In this manner, there is a reasonable expectation that integrated water resources management will be more effective and efficient compared with a non-integrated approach. However, in promoting a holistic approach, integrated water resources management can experience tension with arrangements for including participatory mechanisms. Many individuals, communities or stakeholder groups do not always give attention to the entire system, but rather only to that part or aspect related to their own needs and interests. Thus, individuals often focus on the impacts of catchment management on their own property, while municipal governments frequently worry about the area under their responsibility. As a result, if integrated water resources management and participatory methods are to be used together, care must be taken to understand the strengths and limitations of both.

Collaboration allows stakeholders to join forces in sharing their views on different aspects of a problem, and then together explore differences and search constructively for solutions going beyond any one stakeholder's capacities and limitations. In this way, they can share resources, enhance each other's capacity for mutual benefit and achieve a common purpose by sharing risks, responsibilities and rewards (Gray, 1989; Himmelman, 1996).

In addition to the above features, Gunton and Day (2003) point out that it is essential to determine if a collaborative approach is appropriate in any specific situation. In their view, a collaborative approach "may not work in all circumstances" . To help determine when participatory approaches are appropriate, they identify five pre-conditions for success:

(a) Commitment of decision-making agencies to a participatory approach;
(b) Commitment of all stakeholders;
(c) Urgency for resolution of an issue or issues;
(d) Absence of fundamental value differences;
(e) Existence of feasible solutions. In their view, the challenge is not whether all pre-conditions are met perfectly, but whether they are met adequately enough to allow a participatory process to begin.

3.6.5 **Links to regional planning and impact assessment**

Integrated water resources management plans or strategies often lack a legislative or statutory basis. This can have several negative consequences. First, agencies receiving recommendations from an integrated water resources management plan may simply ignore them, believing that they fall outside their legislated mandate or mission. Second, if agencies do strive to implement recommendations from such a plan, they have to determine what priority these recommendations should have relative to other responsibilities. For either of these reasons, there is a high probability that little action will be taken.

One way to overcome this problem is to link recommendations to instruments – such as official regional or municipal land-use plans, or environmental impact assessments – which have a statutory basis. It was for that reason that in 3.5.3 the discussion highlights how recommendations from integrated water resources management catchment, sub-catchment, tributary or environmental site plans in Ontario were incorporated into official plans, secondary plans or environmental impact assessment processes.

Water planners and managers should therefore familiarize themselves with the opportunities to connect the recommendations from integrated water resources management plans to regional or local land-use official plans or to environmental impact assessment processes, when these have a statutory basis. Another alternative is to strive for a statutory basis for integrated water resources management but, at the moment, such arrangements are the exception, not the rule.

3.6.6 Designing institutional arrangements

Once a vision is established, it is important to consider the institutional arrangements – the formal and informal mix of values, rules, organizational structures and cultures, mechanisms and processes – available for implementing integrated water resources management.

Experience suggests that governments often look first to make changes to organizational structures, such as when ministries of the environment were created in the 1970s or ministries of sustainable development, in the 1990s. However, this can be effective only where edge or boundary problems are identified, as highlighted in 3.2.6, and are therefore rarely the best place to start. As a result, when introducing or modifying institutional arrangements for integrated water resources management once a vision has been established, planners and managers should do the following:

(a) Determine what actions can be taken to give credibility or legitimacy to integrated water resources management. This is usually done by having some combination of a legislative base, administrative policy commitment and ongoing financial support;
(b) Decide which management functions are to be integrated. Given their utility-like characteristics, some functions, such as water supply, sewage treatment and waste disposal, could be allocated to the private sector, while others, such as flood-plain management or wetland protection, should be allocated to the public sector on the basis of their common property characteristics;
(c) Determine appropriate organizational structures, on the principle that structures should follow, not lead, functions. A continuum of structures exists, ranging from one large, centralized, multiple-purpose organization to many, small, decentralized, single- or limited-function organizations. Each arrangement has strengths and weaknesses and all encounter edge or boundary problems;
(d) As structures will never align perfectly with functions, next consider what mix of processes – for example, public participation and impact assessment – and mechanisms, such as interdepartmental task forces or committees, will be most effective to ensure coordination, collaboration and coherence among different agencies or groups. The following observation has been made (Grindle and Hilderbrand, 1995):

> Capacity builders need to create active mechanisms for interaction and coordination. Formal means of communication and coordination can be created, such as high-level and technical-level coordination committees, interlocking boards of directors or advisors, joint workshops and seminars, and relocating offices or improving technology so that communication is physically easier. Informal communications can be stimulated to supplement and support these formal interactions.

The value of such initiatives has been reiterated by noting that:

> interventions which allow professionals to work alongside one another as equals are increasingly important. Such interventions include networking and twinning arrangements, as well as workshops, seminars and platforms for cooperation which facilitate the sharing of knowledge (Franks, 1999);

(e) Most importantly perhaps, managers and planners should establish organizational cultures and staff attitudes to foster collaboration and cooperation, rather than competition. According to Grindle and Hilderbrand (1995), "Without exception, the organizations that performed well were able to inculcate a sense of mission and commitment to organizational goals among staff" and "one of the most important sets of findings is the evidence that relates organizational performance to the strength and orientation of its organizational culture". Such a

supportive culture can be created and nurtured through training and education programmes focusing on the nature of and need for collaborative processes, and conflict resolution.

The above concepts together provide a framework to assist planners and managers as alternatives are considered regarding appropriate institutional arrangements to support integrated water resources management.

3.6.7 Monitoring and evaluating

As noted in 3.5.2, it is important to have a desirable future for which to aim and then use integrated water resources management as a means to achieve it. It is equally important to include provision for monitoring and evaluation so that the journey toward a desirable future can be tracked and, if necessary, adjusted.

To ensure a results-based focus, it is normal to monitor for effectiveness: are objectives being achieved? In addition, attention should be given to efficiency (are objectives being attained in the most cost-effective manner?) and equity (are the benefits and costs being distributed fairly?). Another dimension increasingly receiving attention is transparency or accountability: is it possible to see how decisions are taken and resources allocated?

Comparing the above product- and process-oriented dimensions provides a systematic basis against which to assess progress, or lack of progress, related to the role of integrated water resources management in helping achieve a vision. Without systematic monitoring followed by evaluation, the opportunity to learn from experience is reduced, as well as the opportunity to make adjustments in the light of new information, knowledge and experience.

3.7 CAUTIONS REGARDING INTEGRATED WATER RESOURCES MANAGEMENT

3.7.1 When to apply integrated water resources management

It is all too often assumed that integrated water resources management is good or desirable. However, because integration does not occur without costs, care should be taken when deciding whether or not integrated water resources management is appropriate. Staff time and other resources are required to accomplish integration; those resources are then not available for other needs or tasks. Often overlooked is the need to establish that serious resource scarcity and/or environmental degradation problems are the result of many interconnected causal factors whose resolution requires an integrated approach. In contrast, many situations are characterized by relatively straightforward problems that can be handled effectively by one agency or organization. If such a situation exists, integrated water resources management is unlikely to be needed. However, if there are multiple causes, or the actions of numerous agencies or participants might work at cross-purposes or could be designed to complement each other, then integrated water resources management will be appropriate (Hooper and others, 1999).

3.7.2 Implementation gap

Once it has been decided that integrated water resources management is appropriate, it is important to ensure that capacity exists to move from concept to action. As indicated in 3.5, many challenges can be encountered when striving to implement integrated water resources management: too broad an interpretation which leads to difficulties in completing analyses and plans in a timely manner, lack of a vision to be achieved through use of integrated water resources management, lack of recognition of the need to change the detail sought as the spatial scale changes, confusion over the role of partners or stakeholders, lack of credibility or legitimacy of a integrated water resources management plan, inadequate institutional arrangements and low monitoring and evaluation capacity. Any one or a combination thereof can hinder integrated water resources management. Most of these aspects are not unique to the approach, but are generic challenges for planning and management. Nevertheless, if these shortcomings are not recognized and addressed, they will most likely contribute to integrated water resources management being ineffective, and thus to its being discredited.

References and further reading

Burton, J.R., 1986: The Total Catchment Management concept and its application in New South Wales. Proceedings of the Hydrology and Water Resources Symposium, 1986. Brisbane, Queensland, Griffith University.

Department of Water Affairs and Forestry, 1997: White Paper on a National Water Policy. Department of Water Affairs and Forestry, Pretoria.

Department of Water Affairs and Forestry, and Water Research Commission, 1996: The Philosophy and Practice of Integrated Catchment Management:

Implications for Water Resource Management in South Africa. Discussion Document, WRC Report No. TT 81/96, Pretoria, Water Research Commission.

Görgens, A., G. Pegram, M. Uys, A. Grobick, L. Loots, A. Tanner and R. Bengu, 1998: Guidelines for Catchment Management to Achieve Integrated Water Resource Management in South Africa. WRC Report No. KV 108/98, Pretoria, Water Research Commission.

Gray, B., 1989: *Collaborating: Finding Common Ground for Multiparty Problems*. San Francisco, Jossey-Bass. Abstract: http://www.colorado.edu/conflict/peace/example/gray7278.html.

Grindle, M.S. and M.E. Hilderbrand, 1995: Building sustainable capacity in the public sector: what can be done? *Public Administration and Development*, 15:441–463. Abstract: at http://www.grc-exchange.org/info_data/record.cfm?ld=35.

Gunton, T.I. and J.C. Day, 2003: The theory and practice of collaborative planning in resource and environmental management. *Environments*, 31(2): 5–19. Abstract: http://goliath.ecnext.com/coms2/summary_0199-1243923_ITM&referid=2090.

Himmelman, A.T., 1996: On the theory and practice of transformational collaboration: from social service to social justice. In: *Creating Collaborative Advantage* (E. Huxham, ed.). Thousand Oaks, California, SAGE Publications.

Hooper, B.P., G.T. McDonald and B. Mitchell, 1999: Facilitating integrated resource and environmental management: Australian and Canadian perspectives. *Journal of Environmental Planning and Management*, 42:747–766.

Inter-American Development Bank, 1998: *Strategy for Integrated Water Resources Management*, Publication No. ENV-125. Washington, DC, Inter-American Development Bank.

Intergovernmental Panel on Climate Change (IPCC), 2007: Fourth Assessment Report – Climate Change 2007. Geneva.

International Conference on Water and the Environment, 1992: International Conference on Water and the Environment: Development Issues for the 21st Century, Keynote Papers. Geneva, ICWE Secretariat, World Meteorological Organization.

International Union for Conservation of Nature/United Nations Environment Programme/World Wildlife Fund, 1991: *Caring for the Earth: a Strategy for Sustainable Living*. Gland.

Jenkins, H.A., 1976: *A Valley Renewed: The History of the Muskingum Watershed Conservancy District*. Kent, Ohio, Kent State University Press.

Lazarus, P. 1997: Towards a Regulatory Framework for the Management of Groundwater in South Africa. WRC Report No. 789/1/98. DWAF Report Geo 2.2(389). Pretoria, Water Research Commission and Department of Water Affairs and Forestry.

MacKenzie, S.H., 1996: *Integrated Resource Planning and Management: The Ecosystem Approach in the Great Lakes Basin*. Washington, DC, Island Press.

Memon, P.A., 2000: Freshwater management policies in New Zealand. In: Environmental Planning and Management in New Zealand, (P.A. Memon and H. Perkins, eds). North Palmerston, New Zealand, Dunmore Press. Abstract: at http://www3.interscience.wiley.com/cgi-bin/abstract.14322/ABSTRACT.

Mitchell, B. and D. Shrubsole, 1992: *Ontario Conservation Authorities: Myth and Reality*, Department of Geography Publication Series No. 35. Waterloo, Ontario, University of Waterloo.

Mitchell, B. (ed.), 1990: *Integrated Water Management: International Experiences and Perspectives*. London, Belhaven Press.

Organisation for Economic Cooperation and Development, 1989: *Water Resource Management: Integrated Policies*. Paris, Organisation for Economic Cooperation and Development.

Richardson, A.H., 1974: *Conservation by the People: The History of the Conservation Movement in Ontario to 1970*. Toronto, University of Toronto Press.

Tortajada, C., 2003: Workshop on Integrated Water Resources Management for South and South-East Asia, Bangkok, Thailand, 2–4 December 2002. *Water International,* 28(1)130–131.

United Nations, 1977: Report of the United Nations Water Conference, Mar del Plata, 14–25 March 1977, E/CONF.70/29. New York, United Nations.

———, 1992: Report of the United Nations Conference on Environment and Development, Rio de Janeiro, 3–14 June 1992. United Nations publication E.93.I.8, Three Volumes. New York, United Nations.

———, 1997: *Comprehensive Assessment of the Freshwater Resources of the World*. New York, United Nations.

Van Zyl, F., 1999: Above all else what do we have to do make a sustainable impact on diffuse source pollution? Proceedings of the International Conference on Diffuse Pollution. C. Barber, B. Humphries and J. Dixon (eds). Wembley, Western Australia, CSIRO Land and Water.

World Summit on Sustainable Development, 2002: Plan of Implementation, at http://www.johannesburgsummit.org.html/documents/summit_docs/2309_planfinal.html.

Young, G.J., J.C.I. Dooge and J.C. Rodda, 1994: *Global Water Resource Issues*. Cambridge, Cambridge University Press.

CHAPTER 4

APPLICATIONS TO WATER MANAGEMENT

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,

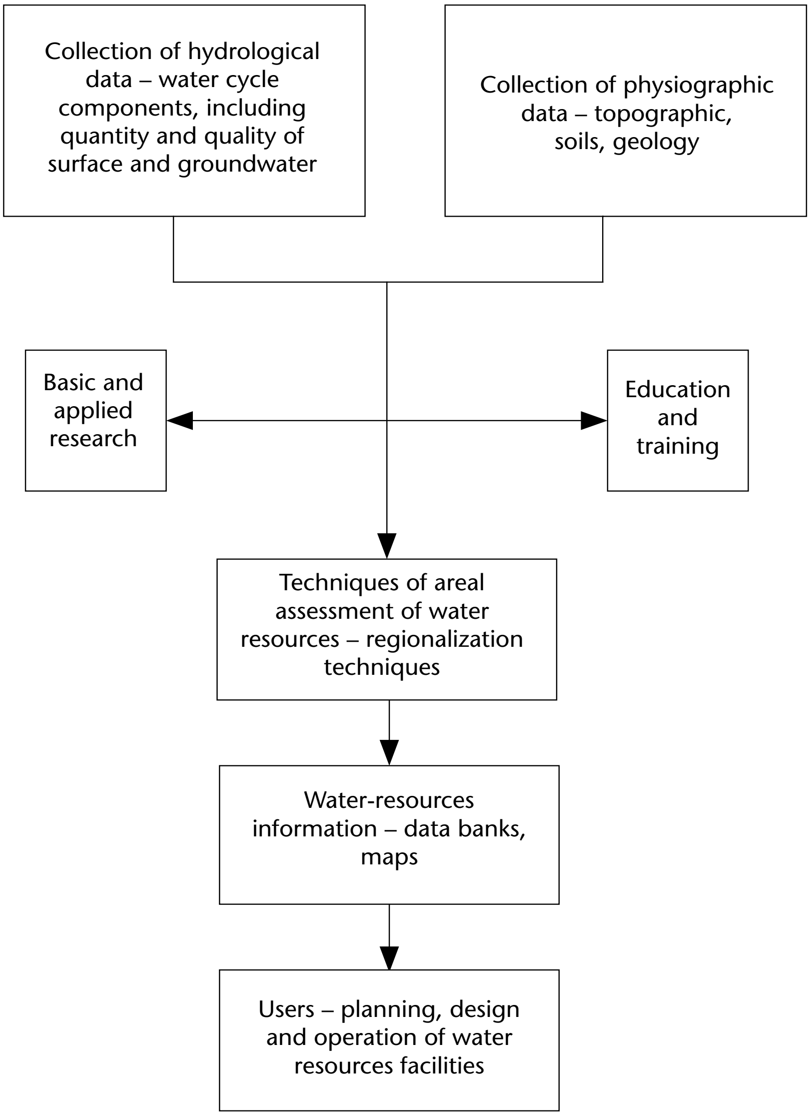


Figure II.4.1. Components of a basic water resource assessment programme

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.

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 socioeconomic 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?

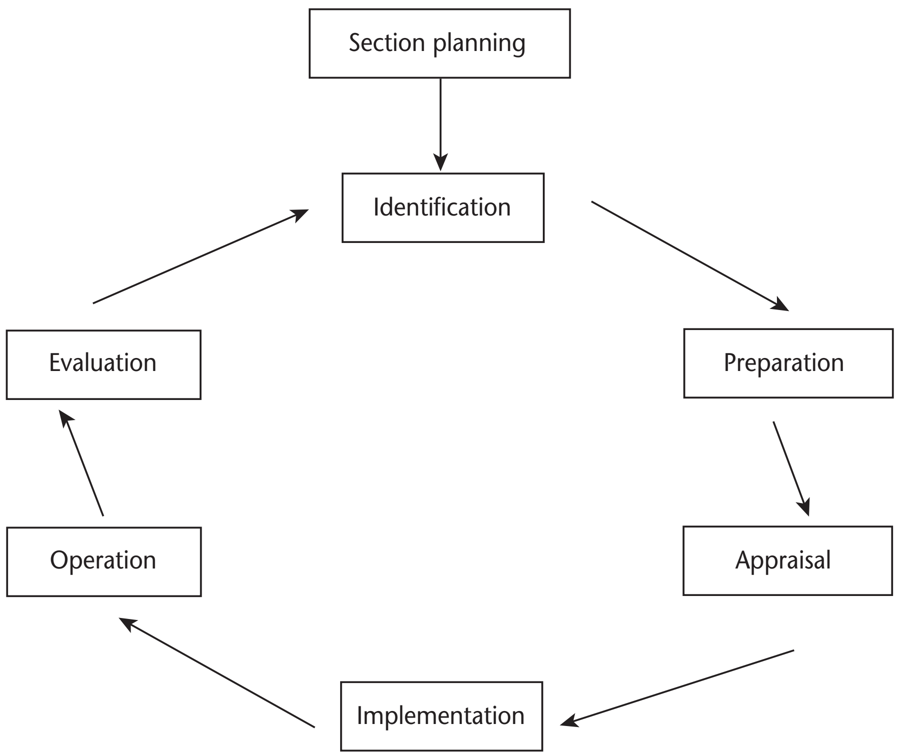


Figure II.4.2. Project cycle

(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

Purpose	*Features*	*Concern*	*Required data*
Reconnaissance		Hydrology	Drainage network Watersheds Springs Distinction of perennial from intermittent and ephemeral streams
		Physiography	Geology Topography and morphology Soil cover and types Urbanization
		Meteorology	10, 11 Temperature distribution Wind distribution Snowpack distribution
		Streamflow	1, 2, 3, 4, 7, 8, 9 – at selected sites
		Floods	4, 5, 6
		Groundwater	12, 13
Flood control	Structures	Water level	Depth–discharge relationship for important points Hydraulic–topographic relationships in the flood plain 4, 5, 6, 8 Flood-plain occupancy
		Rainfall	Statistics of heavy rainfall in the general area under consideration Pairs of floods and their causing precipitation
	Flood warning	Forecast	Travel times of floods Time lag between precipitation and runoff Flood synchronization at different tributaries
		Prediction	Time series of floods Time series of heavy precipitation
	Flood zoning and insurance	Flood extent	Area–duration–frequency of floods Scour and sedimentation by floods

Purpose	*Features*	*Concern*	*Required data*
Irrigation	Demand	Precipitation	10
		Evapotranspiration	11 Transpiration
		Soil moisture	Soil type Groundwater level
	Supply	Streamflow	1, 2, 3, 4, 7, 8, 9
		Groundwater	12,13
		Reservoir	1, 2, 3, 4, 5, 6, 8, 9, 10, 11
Groundwater management, including recharge	Aquifers	Groundwater	12,13
	Reservoirs and ponds	Streamflow	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11
	Bank infiltration	Streamflow	3, 4, 6, 7, 8, 9
	Wells	Streamflow	1, 2, 3, 4, 5, 6, 8, 9, 10, 11
Power generation	High-head dams	Streamflow	1, 2, 3, 4, 5, 6, 8, 10, 11
	Low-head dams	Streamflow	2, 3, 4, 6, 7, 8 Tailwater depth–discharge relationship
Navigation	Channels	Water depth	Depth–discharge relationship for important points 2, 3, 7, 8
		Flood flows	4, 6 Rates of high water rise Time lag between rises at different points along the streams Time lag from heavy precipitation to high water Snowmelt distribution
Municipal supply	Rivers	Streamflow and springflow	1, 2, 3, 4, 7, 9
	Reservoirs	Streamflow	1, 2, 3, 4, 5, 6, 8, 9, 10, 11
		Groundwater	12,13
Industrial use	Rivers	Streamflow	1, 2, 3, 4, 7, 8, 9
	Reservoirs	Streamflow	1, 2, 3, 4, 5, 6, 8, 9, 10, 11
	Aquifers	Groundwater	12,13

Purpose	*Features*	*Concern*	*Required data*
Recreation, aesthetics and tradition	Lakes and reservoirs	Physiography	Storage–elevation relationship Shoreline properties Wave possibilities 9
		Climate	7, 10, 11 Air temperature distribution Wind distribution
	Rivers	Streamflow	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11
		Physiography	Channel geometry Depth–discharge velocity relationships Bank soil and cover
		Streamflow	2, 3, 4, 6, 7, 8, 9 Reservoir release variations
Salinity and sediment control	Dilution	Streamflow	2, 3, 4, 6, 7, 8, 9
	Cleaning	Floods	4, 6, 8, 9
		Reservoirs	1, 2, 3, 4, 5, 6, 8, 9, 10, 11
Pollution abatement	Dilution	Streamflow	1, 2, 3, 4, 7, 8, 9
	Cleaning	Floods	4, 6, 9
		Reservoirs	1, 2, 3, 4, 5, 6, 8, 9, 10, 11
Fish and wildlife conservation	Rivers	Streamflow	2, 3, 4, 6, 7, 8, 9
		Lakes and reservoirs	Water level fluctuation distribution 9
		Structures	Resulting changes in water depth, velocity, temperature, sediment load and bank characteristics, upstream and downstream

Note: Numbers refer to common hydrological data listed below.

1 – Series of monthly and annual volume of streamflow
2 – Mean daily discharge series
3 – Low-flow frequency distribution
4 – Frequency distribution of high discharges
5 – Frequency distribution of large-volume floods
6 – Shapes of typical flood hydrographs
7 – Ice cover information
8 – Erosion, sediment transport and deposition
9 – Water quality
10 – Precipitation distribution in space and time
11 – Evaporation distribution in space and time
12 – Aquifer extent and characteristics
13 – Series of water levels of relevant aquifers

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

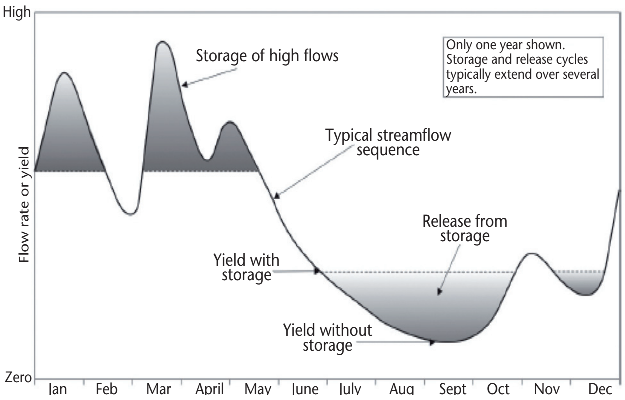


Figure II.4.3. Streamflow and storage

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 underlaid 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:

$$S_i = S_{i-1} + I_i + P_i - E_i - D_i - O_i = S_{i-1} + \Delta S_i \qquad (4.1)$$

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 ΔS_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:

$$S_t = S_0 + \int_0^t (I - D)\,d\tau = S_0 + \int_0^t I\,d\tau - \int_0^t D\,d\tau = S_0 + I_t^* - D_t^* \qquad (4.2)$$

(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.

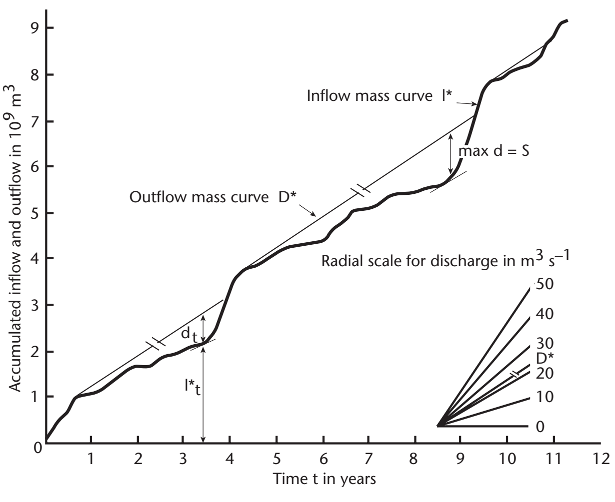


Figure II.4.4. Mass-curve approach for determining reservoir storage capacity

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.

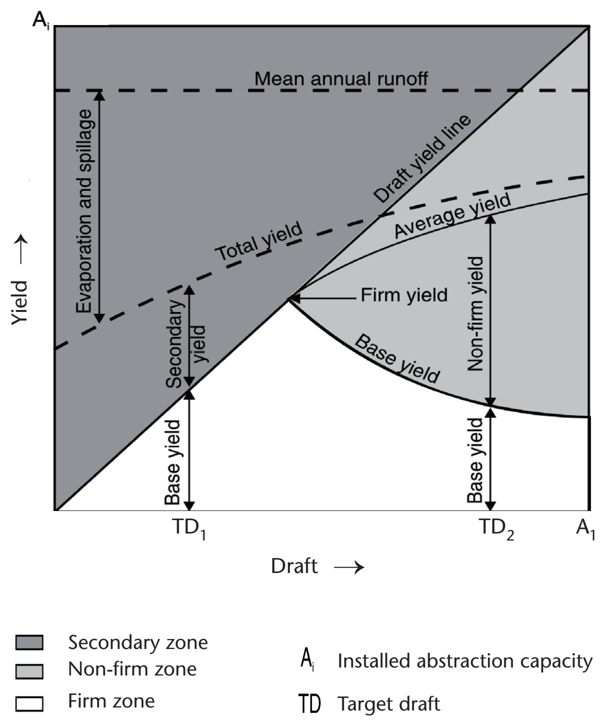


Figure II.4.5. Simplified draft–yield response diagram

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:

$$R_n = R_o + R_{ir} + R_d + R_{gw} - R_{ri} - R_{rd} - R_{rg} + S + E \quad (4.3)$$

where R_n is the natural flow, R_o is the observed flow, R_{ir} 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_n;

(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:

$$R = 1/T \quad (4.4)$$

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

failure. Annual reliability of supply is therefore related to the recurrence interval of failure by the following relationship:

$$r = 1 - 1/T \qquad (4.5)$$

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:

$$R_n = 1 - (1 - R)^n = 1 - (1 - 1/T)^n \qquad (4.6)$$

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

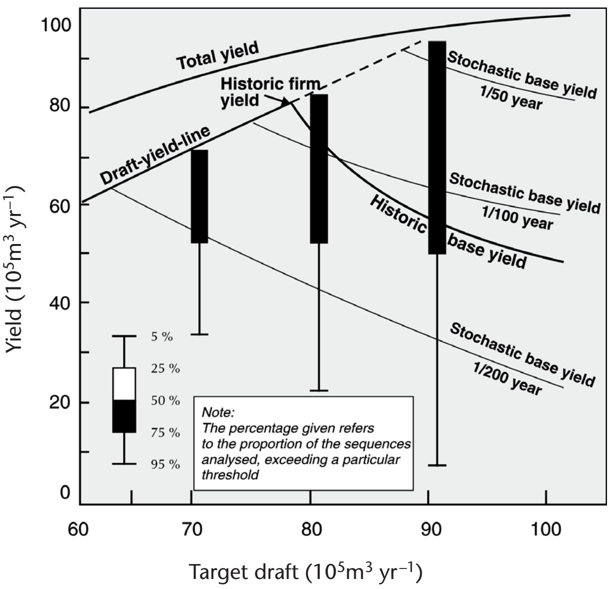


Figure II.4.6. Comparison of long-term yield characteristics

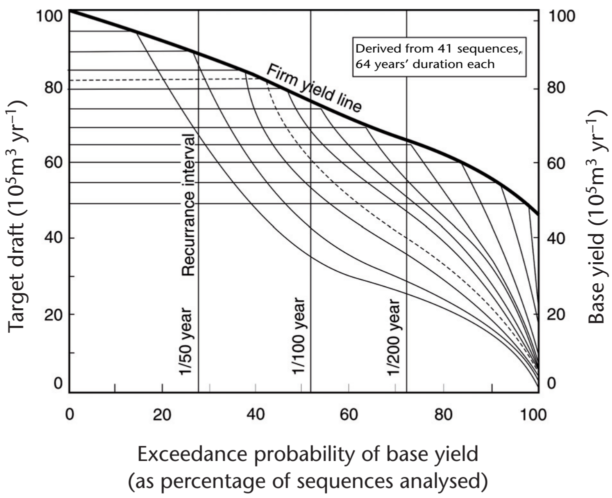


Figure II.4.7. Family of long-term draft–yield reliability characteristics

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 m^3 per year on the system (see dotted line). The additional 22 million m^3 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 m^3 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

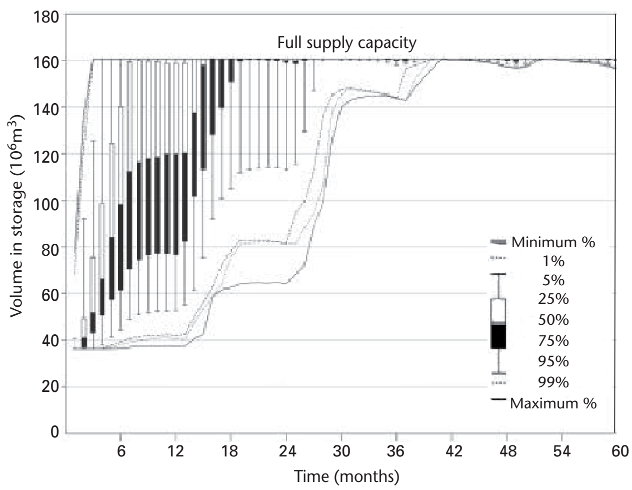


Figure II.4.8. Probabilistic assessment of reservoir filling time

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

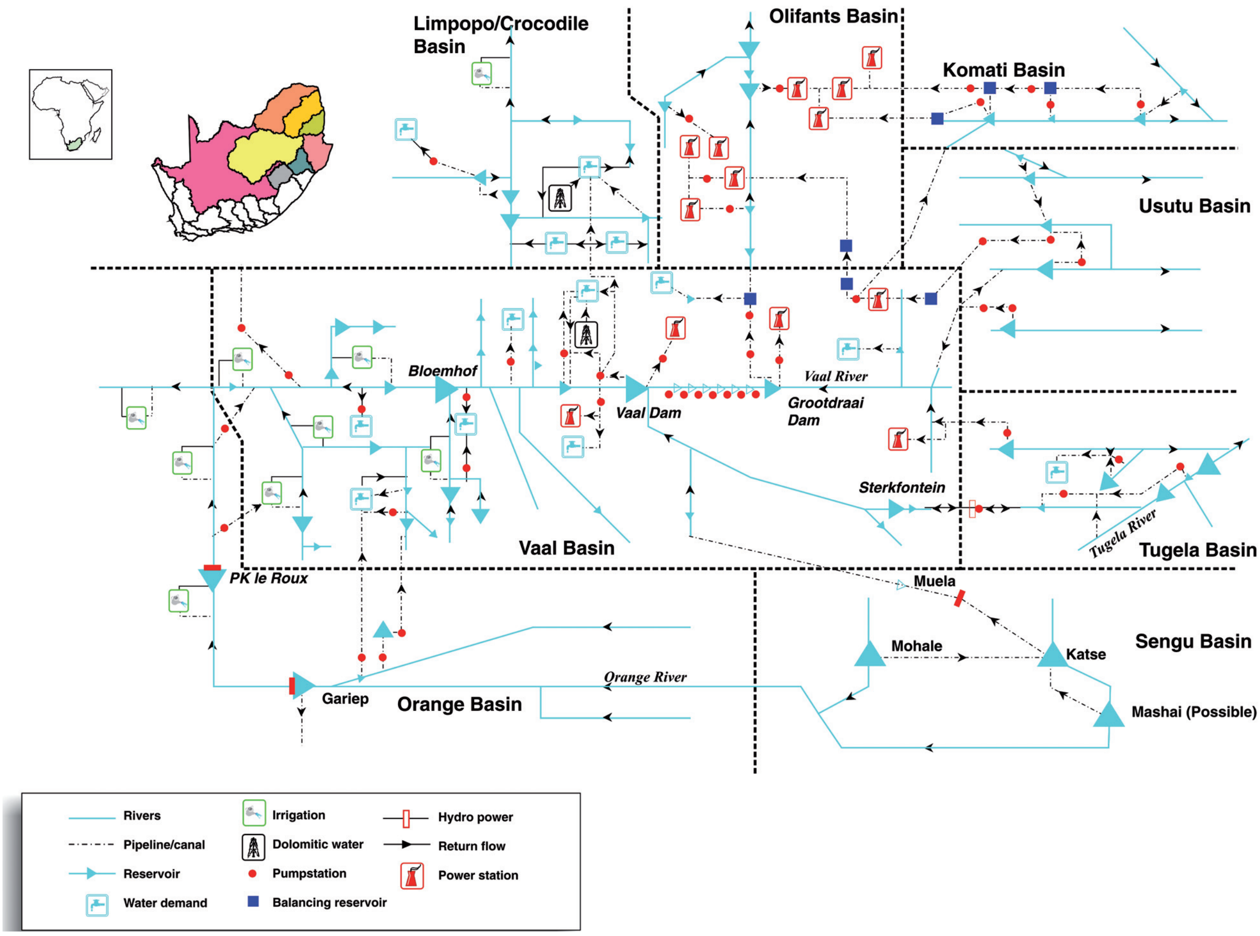


Figure II.4.9. Multi-reservoir water resource system extending over several river basins

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

reservoirs can be used to estimate the volume of water in storage. The procedures that are used to estimate lake volumes depend on an empirical relationship between surface area or shoreline length and volume. Either an area–volume relationship may be developed or topographic features can be used to estimate the water stage in the reservoir and then relate the stage to water volume. Government agencies can use this approach not only to supplement data they obtain for the reservoirs they manage, but also to maintain an awareness of reservoirs they do not control but which may affect their own management strategies under extreme conditions such as major flooding. These techniques clearly demand high spatial resolution data except under extreme flood conditions (Engman and Gurney, 1991).

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 [HOMS 181, J04, J10, J15, K10, K15, K22, K45]

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;

(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, 2006*a*). 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,

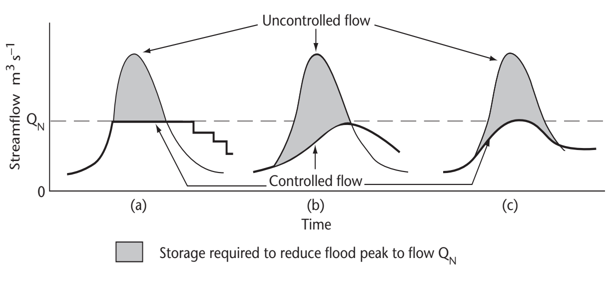


Figure II.4.10. Effects of reservoirs on floods – regulated storage (a), unregulated on-stream storage (b) and unregulated off-stream storage (c)

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.

4.3.4.2.7 *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;
(b) Flood routing of the diverted flow through the diversion channel;
(c) Combination of the diverted flow with floods which may occur in the receiving river;
(d) 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

(6.3.6). An incidental effect of channel modification may be increased scouring in the modified reach and sediment deposition downstream.

Opposite effects can be achieved by reducing the channel capacity by various river-training structures which, by slowing the flow, can cause increased flooding upstream and flood reduction downstream.

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 preparednes 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 km^2. 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, 2006*b*).

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 flow 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 pubic 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 Pruit, 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 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 (ET_0), crop evapotranspiration under standard conditions (ET_c) and crop evapotranspiration under non-standard conditions (ET_c). 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, ET_0. 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, ET_c for any particular crop is determined by multiplying the reference evapotranspiration ET_0, with a coefficient K_c, called the crop coefficient (ET_c/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 evapotranspirational requirement 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

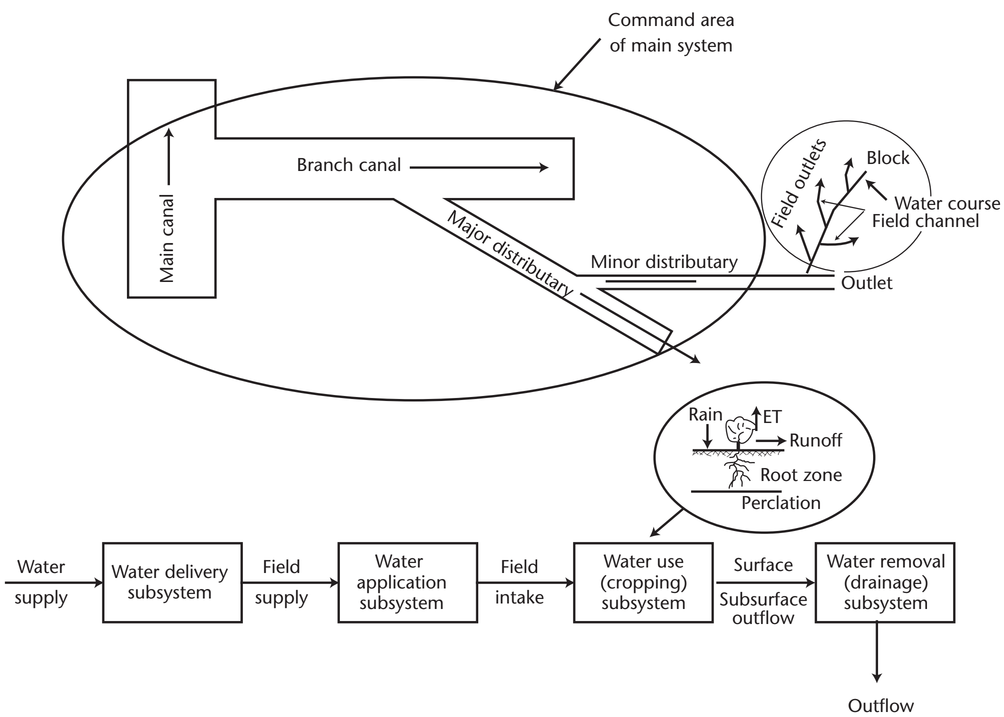


Figure II.4.11. Irrigation system

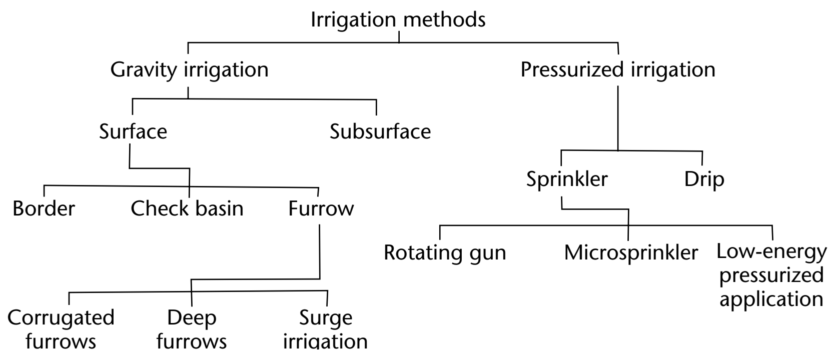


Figure II.4.12. Broad classification of irrigation methods

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.

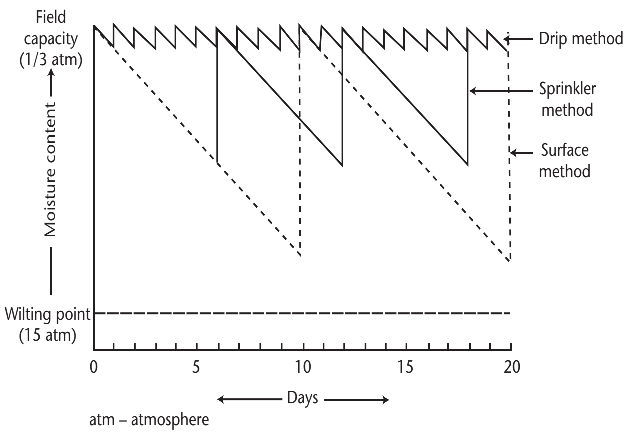


Figure II.4.13. Soil moisture regimes in different irrigation methods

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

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, 1988*a* and 1988*b*; 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 (Doorenboes and Pruit, 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 folllowing:

(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.

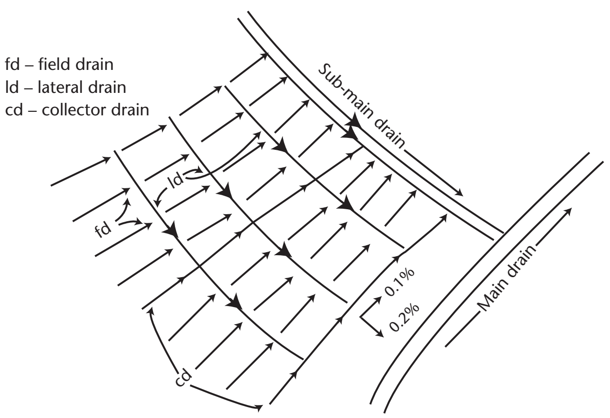


Figure II.4.14. Drainage system layout showing the hierarchy of the components

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 inducement 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.

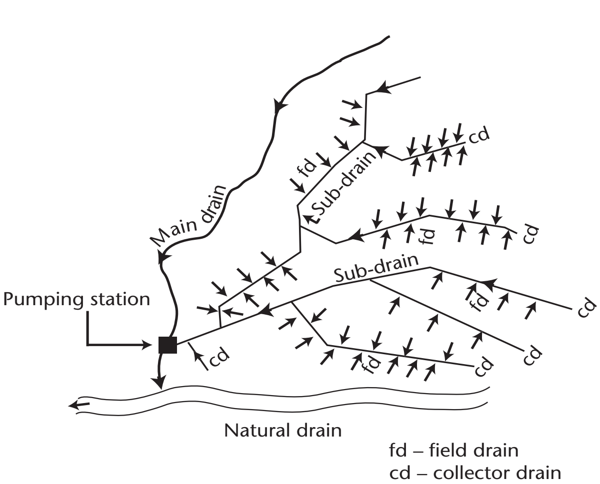


Figure II.4.15. Subsurface drainage system layout

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 thermoelectric 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:

$$P = 9.81 \eta Q H \qquad \text{(in kW)} \qquad (4.7)$$

where η 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:

$$P = 8.5 Q H \qquad (4.8)$$

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

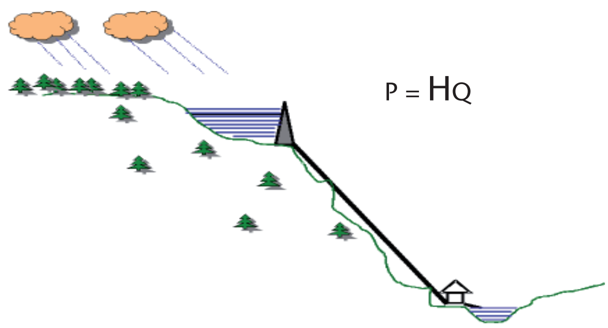


Figure II.4.16. High-head power plant

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:

$$P = 9.81\eta QH \quad (4.9)$$

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:

$$P = 9.81\eta QH \quad (4.10)$$

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.

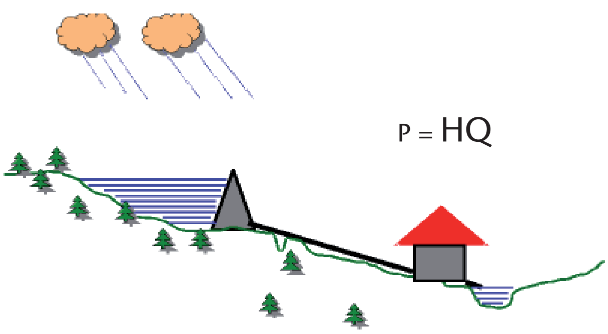


Figure II.4.17. Medium-head power plant

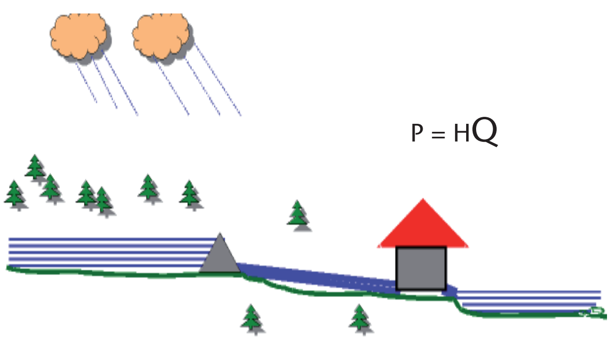


Figure II.4.18. Low-head power plant

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.

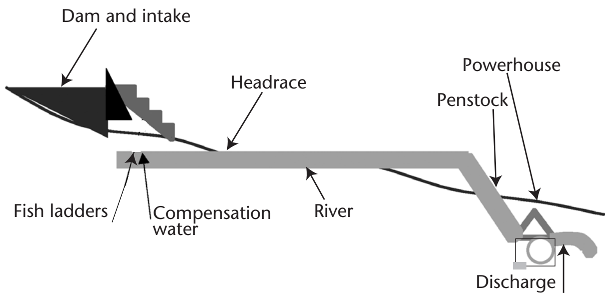


Figure II.4.19. Diagram of a hydropower plant

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

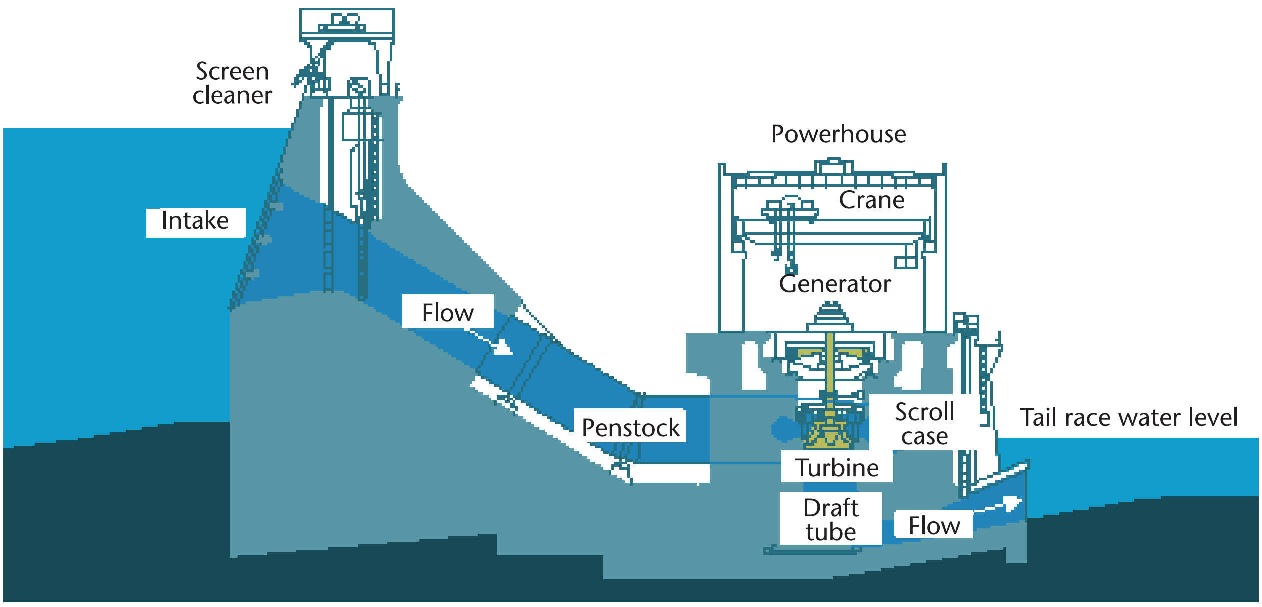


Figure II.4.20. Example of intake in a dam

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. A plan of a typical powerhouse is provided in Figure II.4.22.

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

Figure II.4.21. Example of a partially buried penstock

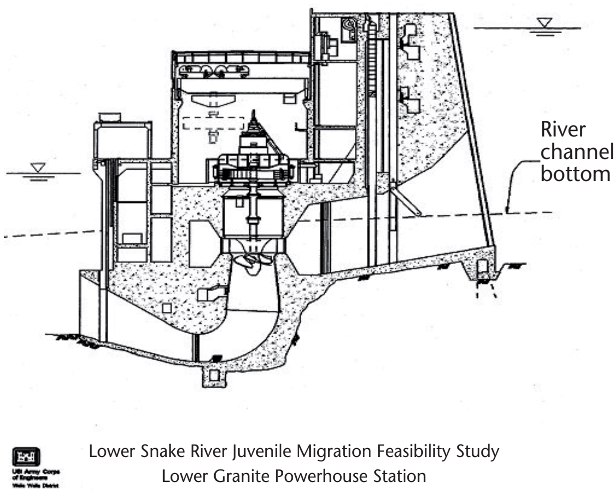


Figure II.4.22. Plan of a typical powerhouse

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

to country regulations or the needs of other water users.

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.

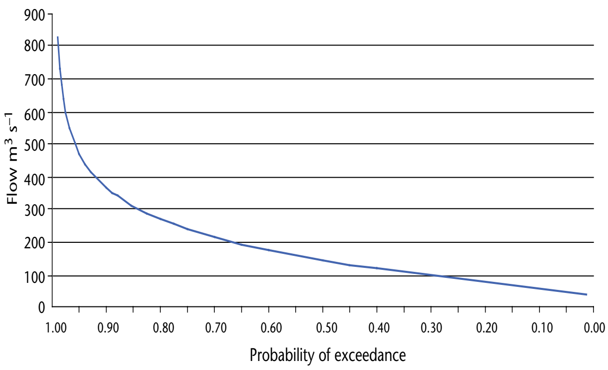


Figure II.4.24. Flow-frequency curve of daily interannual streamflow data

The direct use of this curve is simple, as shown in Figure II.4.25: For a minimum-flow requirement of 50 m^3 s^{-1} (thick line), and a required power plant streamflow of 180 m^3 s^{-1} (dashed line), the flow that can be used by the power plant is limited by these two lines, based on the flow-frequency curve in blue.

In other words, this figure indicates that the power plant will be able to function for 92 per cent of the year, or 336 days a year, when the river discharge is higher than 50 m^3 s^{-1}. However, for 40 per cent of the time, or 146 days a year, the river flow will be greater than can be fed through the turbines; therefore, water will have to be discharged over or through the spillway unless the reservoir has a large enough capacity to store the excess water.

If such a curve is established solely on the basis of monthly mean flows, it can be useful for studying the effect of the dam on high and low flows, but the volume that can actually be used by the turbines will be over-estimated, and in turn the profitability of the system. This error is common and causes serious problems for those who invest in such systems; hence the importance of collecting and using data on mean daily flows.

Furthermore, since these curves are interannual means, they give a picture only of the mean flow on a given day. The usable water volume will vary enormously from year to year, as river flow depends on precipitation, which is not regular from year to year. Therefore, the first years of production might be dry years and the financial amortization might be delayed during the first period. Investors should be aware of these risks and include them in an estimation of overall costs. To this effect, a simulation of the operation of the power station by using daily data of the past several years is often necessary.

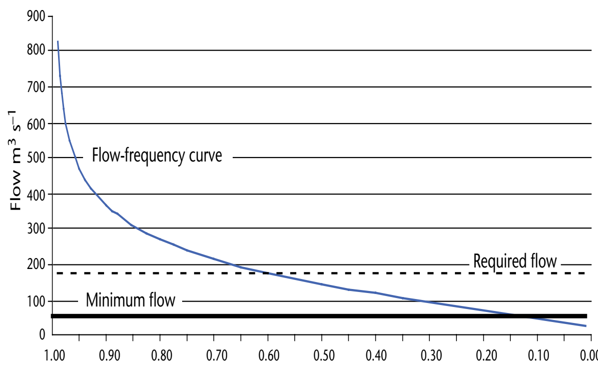


Figure II.4.25. Example of the use of a flow-frequency curve

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:

$$E = \Sigma\, P_i.t_i \qquad (4.11)$$

where E is energy in kWh and P_i 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:

$$E = 8\, AH/3600 \qquad (4.12)$$

where E is the energy in kWh produced during a certain period, A is the volume of usable water in m³ 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 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 spring 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 large 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, irons 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

Type of inland waterways		Classes of navigable waterways	Motor vessels and barges — Type of vessel: General characteristics: Designation	Maximum length	Maximum beam	Draught 7/	Tonnage	Pushed convoys — Type of convoy: General characteristics	Length	Beam	Draught 7/	Tonnage	Minimum height under bridges 2/	Graphical symbols on maps
				L(m)	B(m)	d(m)	T(t)		L(m)	B(m)	d(m)	T(t)	H(m)	
1		2	3	4	5	6	7	8	9	10	11	12	13	14
OF REGIONAL IMPORTANCE	To West of Elbe	I	Barge	38.5	5.05	1.80-2.20	250-400						4.0	
		II	Kampine-Barge	50-55	6.6	2.50	400-650						4.0-5.0	
		III	Gustav Koenigs	67-80	8.2	2.50	650-1,000						4.0-5.0	
	To East of Elbe	I	Gross Finow	41	4.7	1.40	180						3.0	
		II	BM-500	57	7.5-9.0	1.60	500-630						3.0	
		III	6/	67-70	8.2-9.0	1.60-2.00	470-700		118-132	8.2-9.0	1.60-2.00	1,000-1,200	4.0	
OF INTERNATIONAL IMPORTANCE		IV	Johann Welker	80-85	9.5	2.50	1,000-1,500		85	9.5 5/	2.50-2.80	1,250-1,450	5.25 or 7.00 4/	
		Va	Large Rhine vessels	95-110	11.4	2.50-2.80	1,500-3,000		95-110 1/	11.4	2.50-4.50	1,600-3,000	5.25 or 7.00 or 9.10 4/	
		Vb							172-185 1/	11.4	2.50-4.50	3,200-6,000		
		VIa							95-110 1/	22.8	2.50-4.50	3,200-6,000	7.00 or 9.10 4/	
		VIb	2/	140	15.0	3.90			185-195 1/	22.8	2.50-4.50	6,400-12,000	7.00 or 9.10 4/	
		VIc							270-280 1/ 195-200 1/	22.8 33.0-34.2 1/	2.50-4.50 2.50-4.50	9,600-18,000 9,600-18,000	9.10 4/	
		VII						8/	285	33.0-34.2 1/	2.50-4.50	14,500-27,000	9.10 4/	

Figure II.4.27. Example of classification of waterways

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

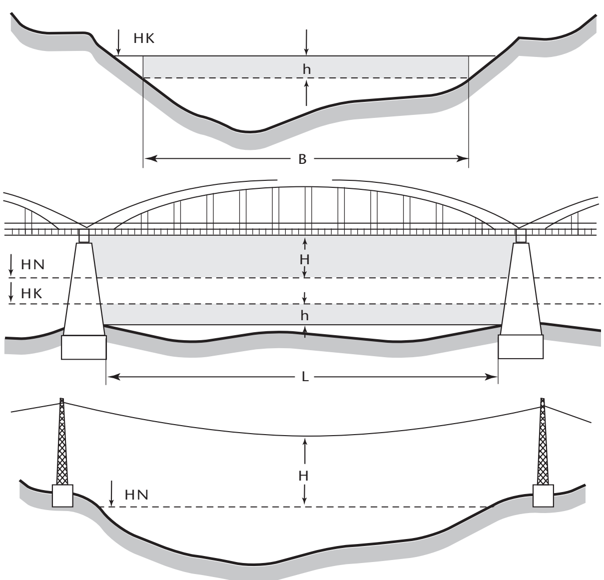
4.6.1.1 Application of hydrological data to the characterization of waterways

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.



Note: See definitions, 4.6.1.1.

Figure II.4.28. Geometrical elements of a waterway

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.

4.6.1.1.1 *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

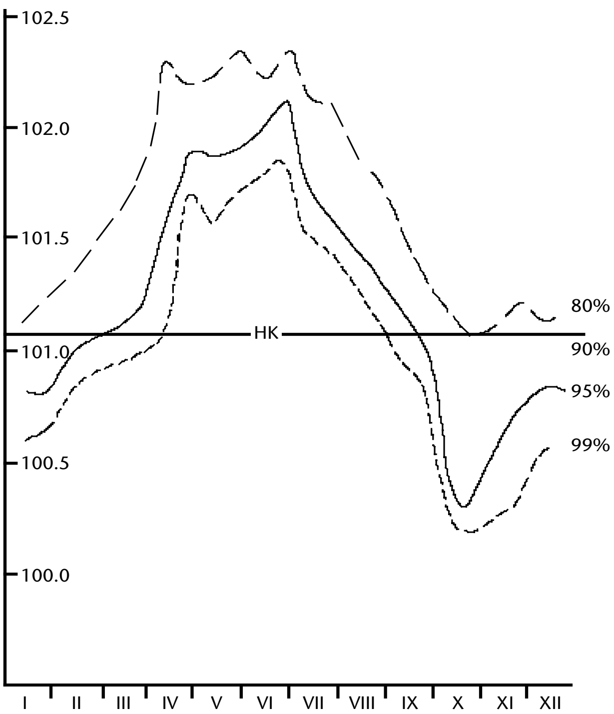


Figure II.4.29. Average flow hydrographs in cross-section at kilometre 1 695 on the Danube

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 icebreakers (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.

4.6.1.1.3 *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

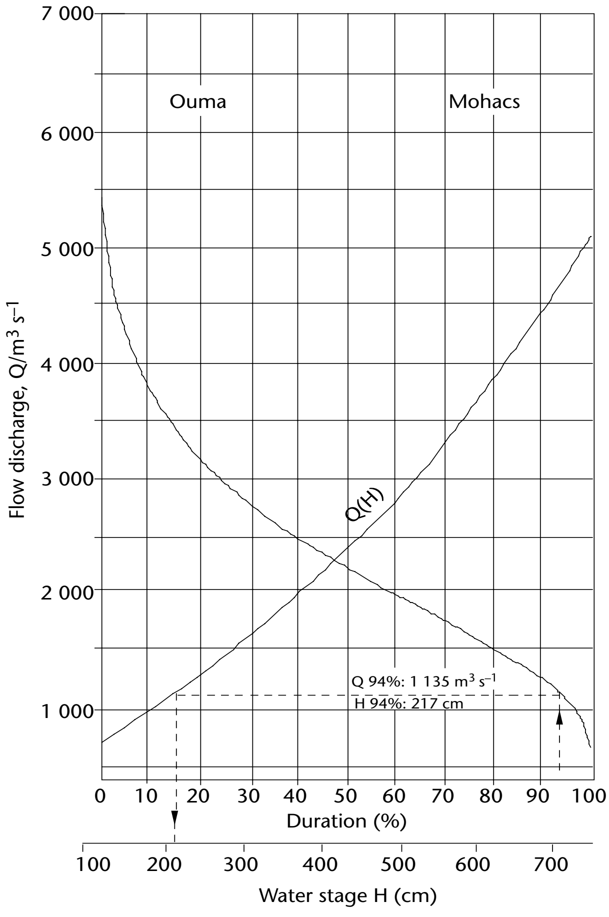


Figure II.4.30. Determination of navigable water stage and flow of a given duration

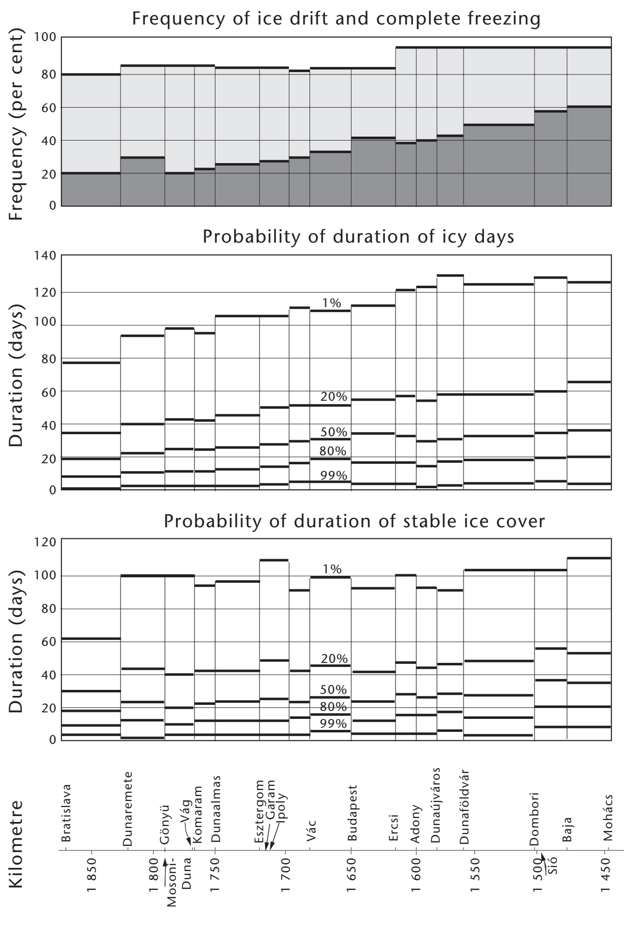


Figure II.4.31. Ice conditions along the Hungarian reach of the Danube river

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:

Probability of exceedance	*Width of confidence band*
60–70%	50 cm
70–80%	40 cm
80–100%	30 cm

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:

$$\tau = \gamma dS \qquad (4.13)$$

where γ 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 γdS.

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).

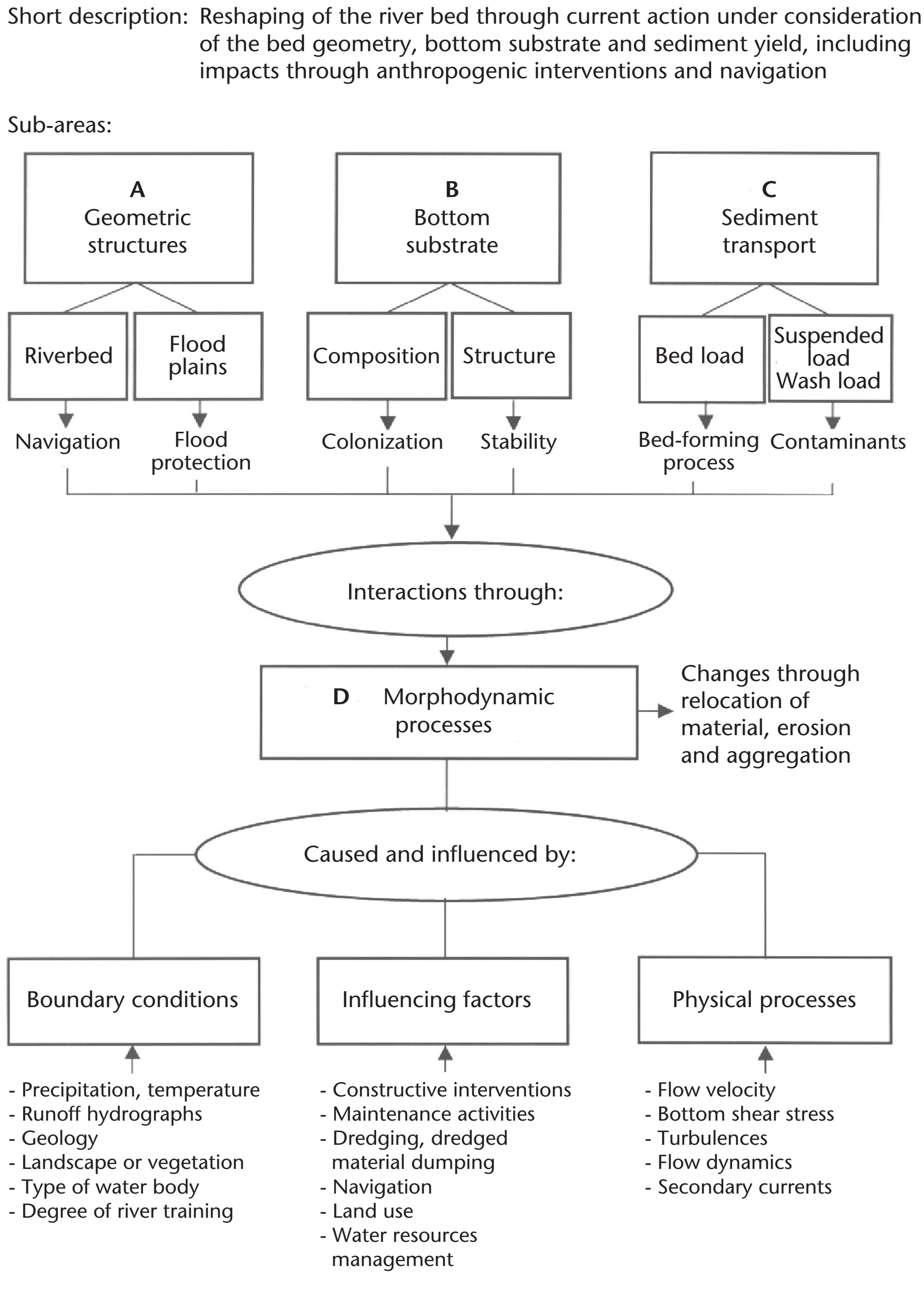


Figure II.4.32. Aspects of river morphology – a schematic overview (German Federal Institute of Hydrology)

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;
α – 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 $\alpha_i < 120°$.

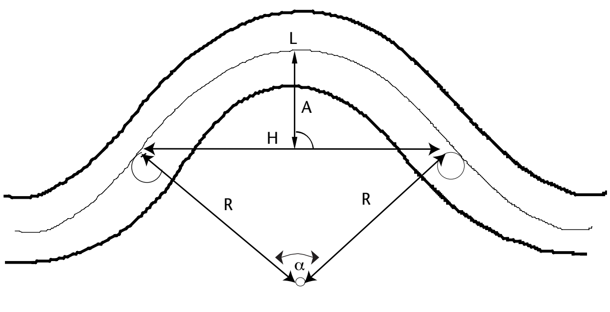


Figure II.4.33. Definition sketch of river bend parameters

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 = F/P);
(e) Average water depth (H = 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.

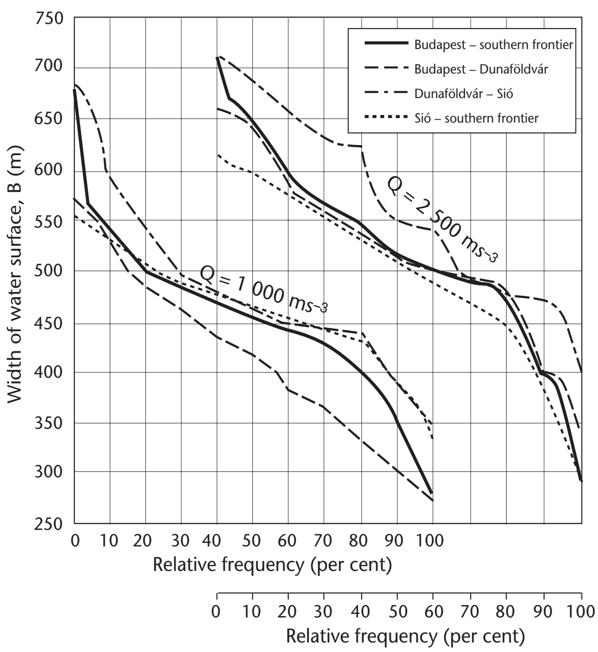


Figure II.4.34. Width variation of the cross-section of the Danube river

Figure II.4.35. A groyne system on the river Elbe, at km 477. The gravel bank on the inside does not obstruct navigation.

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 $P = \Delta f\ Q\ v$ ($m^4\ s^{-2}$), where Δf 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 ΔH_i, 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 $\Delta f_i = \Delta H_i...f_i$ are computed. Finally, for each water stage H_i, the product $P_i = Q_i \cdot v_i \cdot \Delta f_i$ ($m^4\ s^{-2}$) 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_i 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.

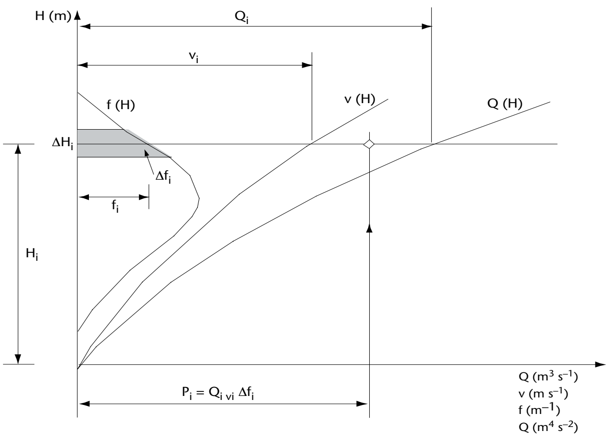


Figure II.4.36. Graphical method used to determine design discharge

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

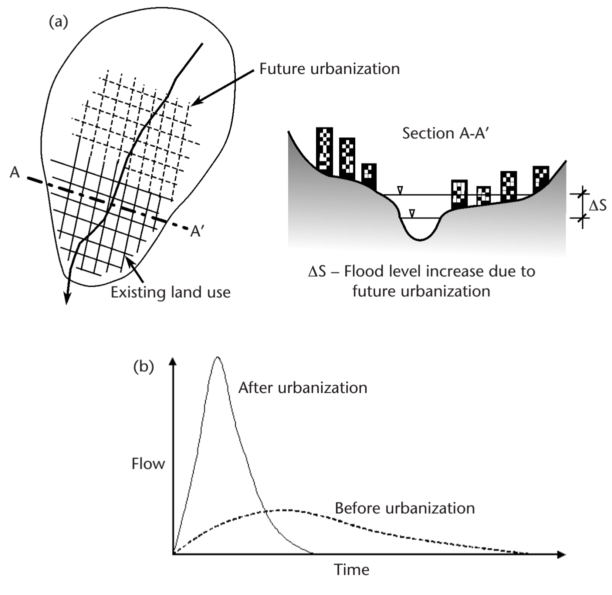


Figure II.4.37. Hydrological impacts of land development: (a) Land-use change causes an increase in flow depth and cross-section and (b) urbanization causes hydrograph change.

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 km^2;
(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)

Constituent (mg/l)	*Residential*	*Mixed*	*Commercial*	*Non-urban*
Biochemical oxygen demand (BOD)	10	7.8	9.3	–
Chemical oxygen demand (COD)	73	65	57	40
Total suspended solids (TSS)	101	67	69	70
Lead (Pb)	0.144	0.114	0.104	0.03
Total copper (cu)	0.033	0.027	0.029	–
Total zinc (Zn)	0.135	0.154	0.226	0.195
Total Kjeldahl nitrogen (TKN)	1.900	1.29	1.180	0.965
Nitrite (NO_2) and nitrate (NO_3)	0.736	0.558	0.572	0.543
Total phosphorus (Tp)	0.383	0.263	0.670	0.121
Soluble phosphorus (Sp)	0.143	0.056	0.080	0.026

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 km^2), 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.

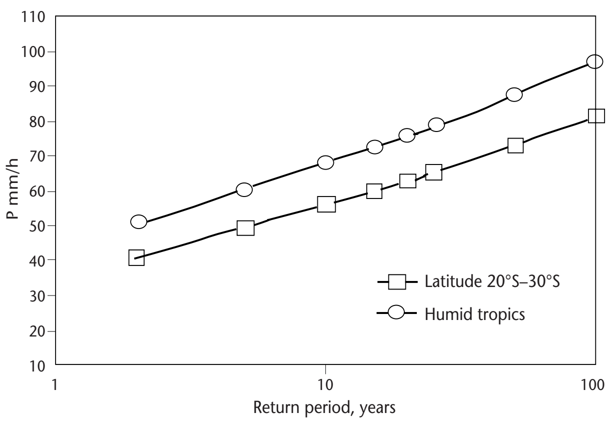


Figure II.4.38. Comparison of the mean maximum rainfall of one-hour duration in the humid tropics and within latitudes 20°S and 30°S in Brazil (Tucci, 2001)

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:

$$AI = 0.489\ DH \qquad (4.14)$$

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 km^2 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:

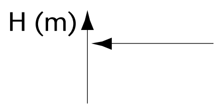
$$Q = 0.278 \cdot C \cdot I \cdot A \qquad (4.15)$$

where Q is the discharge in $m^3\ s^{-1}$, 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):

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

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:



(4.17)

where t_b is in minutes, C_5 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).

The travel time through the system of natural and artificial canals and conduits can be calculated by estimating its velocity using the Manning equation so that:

$$t_r = \sum_{i=1}^{n} \frac{X_i}{V_i} \qquad (4.18)$$

where X_i and V_i are the length of reach i of the system and velocity through it and n is the number of reaches.

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 Cf presented in Table II.4.5.

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

Table II.4.3. Normal range of runoff coefficients (ASCE,1992)

Surface character	*Runoff coefficient C*
Pavement	
– Asphalt and concrete	0.70–0.95
– Brick	0.70–0.85
– Roofs	0.75–0.95
Lawns, sandy soil	
– Flat (2%)	0.05–0.10
– Average (2–7%)	0.10–0.15
– Steep (>7%)	0.15–0.20
Lawns, heavy soil	
– Flat (2%)	0.13–0.17
– Average (2–7%)	0.18–0.22
– Steep (>7%)	0.25–0.35

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

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

Description of the area	*Runoff coefficient C*
Business	
– Downtown	0.70–0.95
– Neighbourhood	0.50–0.70
Residential	
– Single family homes	0.30–0.50
– Multi-units, detached	0.40–0.60
– Multi-units attached	0.60–0.75
– Residential (suburban)	0.25–0.40
– Apartments	0.50–0.70
Industrial	
– Light	0.50–0.80
– Heavy	0.60–0.90
– Parks, cemeteries	0.10–0.25
– Playgounds	0.20–0.35
– Railroad yards	0.20–0.35
– Unimproved	0.10–0.30

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)

Return period in years	*Coefficient Cf*
2–10	1.00
25	1.10
50	1.20
100	1.25

impervious areas. A weighted coefficient can be calculated by:

$$C = C_p + (C_i - C_p)AI \tag{4.19}$$

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):

$$C = 0.05 + 0.9\ AI \tag{4.20}$$

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:

$$C = 0.047 + 0.9\ AI \tag{4.21}$$

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.

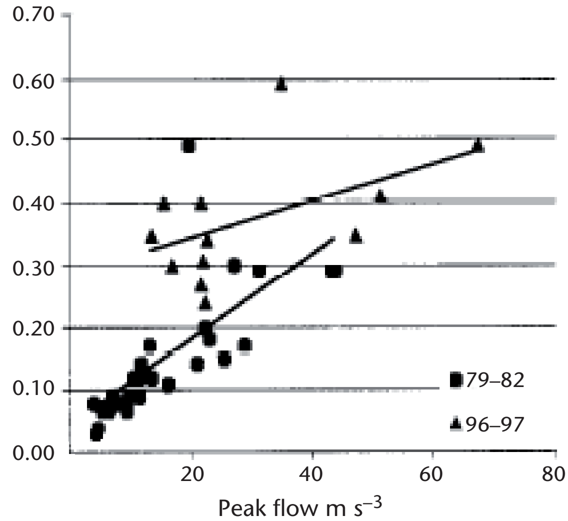


Figure II.4.39. Runoff coefficient (C) for two scenarios: urbanization of 1979–1982 and urbanization of 1995–1997. Diluvio Creek in Porto Alegre (Santos, 1998)

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 Hortonne 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.

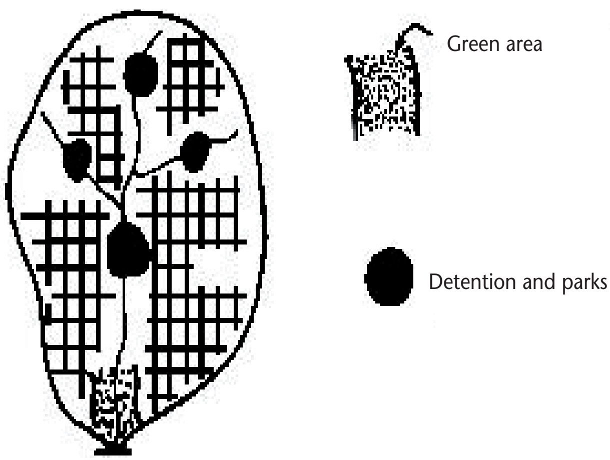


Figure II.4.40. Detention for urban drainage control, planning stage (Tucci, 2001)

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

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
[HOMS I09, K65]

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:

$$A = R \cdot K \cdot LS \cdot P \cdot C \qquad (4.22)$$

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

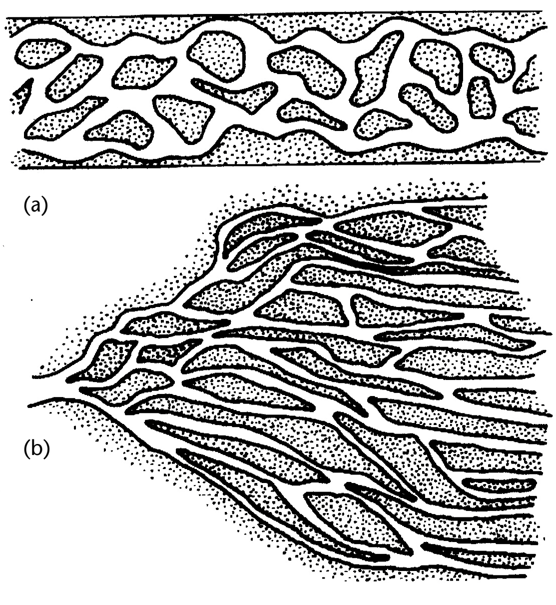


Figure II.4.41. Braided channel (a) and distributary channel (b)

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.

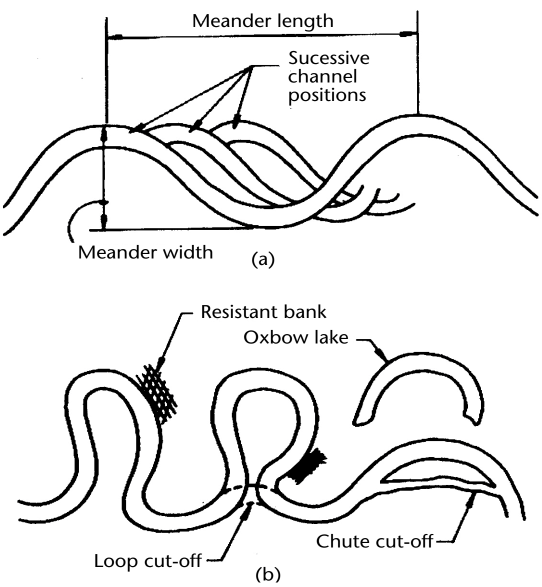


Figure II.4.42. Meander channel (a) and deformed channel (b)

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.

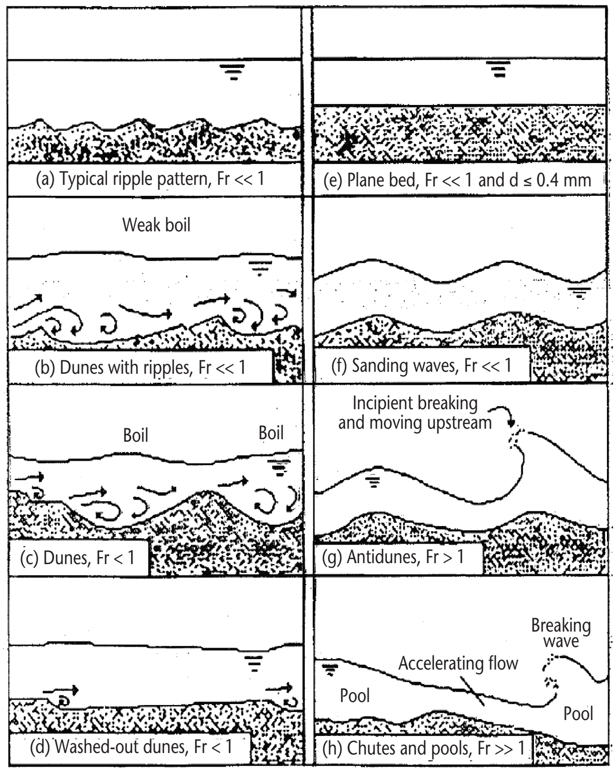
The transition zone encompasses the bed forms that occur during the passage from lower regime to upper regime. This transition is not unique in loose boundary hydraulics. Figure II.4.43 shows bed forms arranged in increasing order of sediment transport rate. The processes concerned are described below (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.



FR – Froude number

Figure II.4.43. Progressive changes in bed-form shape. Adapted from Simons and Richardson (1961) by permission of the American Society of Civil Engineers

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 (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

Bed form	*Regime*	*Approximate Froude number (Fr)*	*Approximate n value*
Ripples	Lower	0.14~0.37	0.018~0.30
Dunes	Lower	0.28~0.65	0.020~0.040
Transitions	Transitions	0.55~0.92	0.014~0.030
Flat bed	Upper	0.70~0.92	0.010~0.030
Antidunes	Upper	>1.0	0.010~0.030

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

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

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:

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

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

This results from the identity:

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

where C is Chezy's coefficient, V is flow velocity, and $V_* = \sqrt{g \cdot R_h \cdot S}$ 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):

$$\log c_s = C \log Q + B \tag{4.26}$$

in which c_s 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:

$$q_s = c\,\frac{\tau_0}{\gamma}\left(\frac{\tau_0}{\gamma} - \frac{\tau_c}{\gamma}\right) \tag{4.27}$$

where q_s is the sediment transport rate per unit width of the channel in kg s^{-1} m^{-1}, $\tau_o = \gamma R_h S_e$ is the shear stress at the channel bed in kg m^{-2}, τ_c is an empirical value for the minimum τ_o required for transporting the sediments considered, γ 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 τ_c parameters

Classification	*Mean diameter (mm)*	*c(kg m^{-3} s^{-1})*	*τ_c(kg/m^2)*
Fine sand	1/8	8 370 000	0.0792
Medium sand	1/4	4 990 000	0.0841
Coarse sand	1/2	2 990 000	0.1051
Very coarse sand	1	1 780 000	0.1545
Gravel	2	1 059 000	0.251
Gravel	4	638 000	0.435

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

$$q_s = \left\{\frac{(\gamma\, q)^{2/3} \cdot S_e - A \cdot d}{B}\right\}^{3/2} \tag{4.28}$$

where q is the water discharge per unit width of the channel in m^2 s^{-1}, γ 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_s 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_{35}, 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):

$$q_s = 8\sqrt{(\beta \cdot \tau^* - 0.047)^3 \; (s-1) g \cdot d^3} \qquad (4.29)$$

where q_s 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 τ^* is the dimensionless boundary shear stress, which is expressed by:

$$\tau^* = \frac{R_h \cdot S}{(s-1)\cdot d} \qquad (4.30)$$

where R_h is the hydraulic radius, S the slope of the channel and s is an adimensional factor, given by $s = \gamma_s/\gamma$, γ_s is the specific weight of the sediment, and γ is the specific weight of the fluid. Finally, the coefficient β 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 $\beta = \left(\frac{K_f}{K_{grains}}\right)^{3/2}$.

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:

$$q_s = 0.05 \cdot \sqrt{(s-1)\cdot g \cdot d^3} \cdot \left(\frac{K_f^2 \cdot R_h^{1/3}}{g}\right) \cdot (\tau^*)^{5/2} \qquad (4.31)$$

where q_s is the rate of sediment transport in m^3 s^{-1} m^{-1}, while s, d, g, K_f, Rh and τ^* 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:

$$q_b = 0.053 \cdot \frac{T^{2.1}}{D_*^{0.3}} \cdot \sqrt{(s-1)\cdot g \cdot d_{50}^3} \qquad (4.32)$$

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:

$$T = \frac{(U_*^2 - U_{*cr}^2)}{U_{*cr}^2} \qquad (4.33)$$

$D_* = d_{50}\ [(s-1)\ g/\nu^2]^{1/3}$, ν is the kinematic viscosity and U_* is the bed shear velocity given by:

$$U_* = \frac{C_f}{C_{grain}}\sqrt{g \cdot R_h \cdot S} \qquad (4.34)$$

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 ranges.

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:

$$\frac{q_{BV}}{\left[g(s-1)d_a^3\right]^{1/2}} = 4 \cdot \left(\frac{d_{90}}{d_{30}}\right)^{0.2} \cdot S^{0.6} \cdot \left(\frac{V}{V_*}\right) \cdot \tau^{*0.5}(\tau^* - \tau_c^*) \qquad (4.35)$$

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_* is shear velocity of flow, τ^* is adimensional shear stress (as in equation 4.29) and τ^*_c, dimensionless critical shear stress, corrected to take into account the slope, expressed by:

$$\tau_c^* = \tau_{0c}^* \cdot \cos\alpha \left(1 - \frac{\tan\alpha}{\tan\varphi}\right) \qquad (4.36)$$

where τ^*_{0c} is the critical Shields parameter, α is the angle of the slope so that $S = \tan\alpha$, and φ 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.

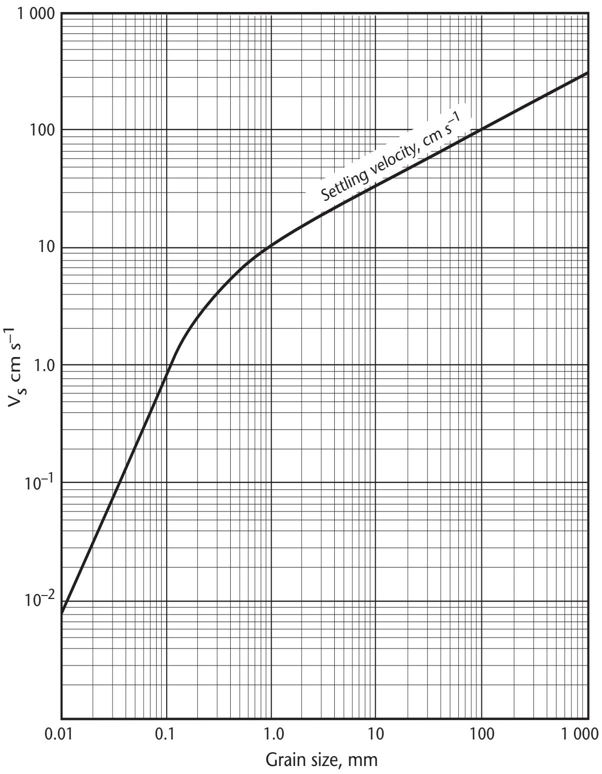


Figure II.4.44. Settling velocity of quartz grains

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

projects, in Brookes and Shields (1996), and water resources in rivers, in Cowx and Welcomme (1998) and WMO/GWP (2006). Thomann and Mueller (1987), and Chin (2006) offer good introductions to the topic of water quality in rivers, lakes and reservoirs.

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.

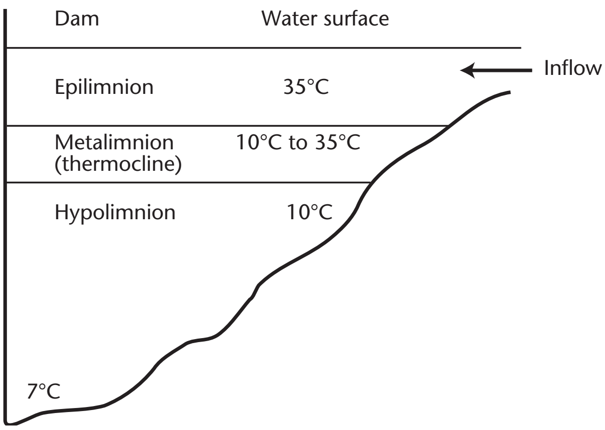


Figure II.4.45. Representative profile showing summer stratification in a large storage reservoir with a high dam

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,

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 polluting matter. Disposal of this material can be a challenge, however;
(c) Forced re-aeration by compressed air in the deoxygenated 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 (1998*a*) 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 ecosystems, 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

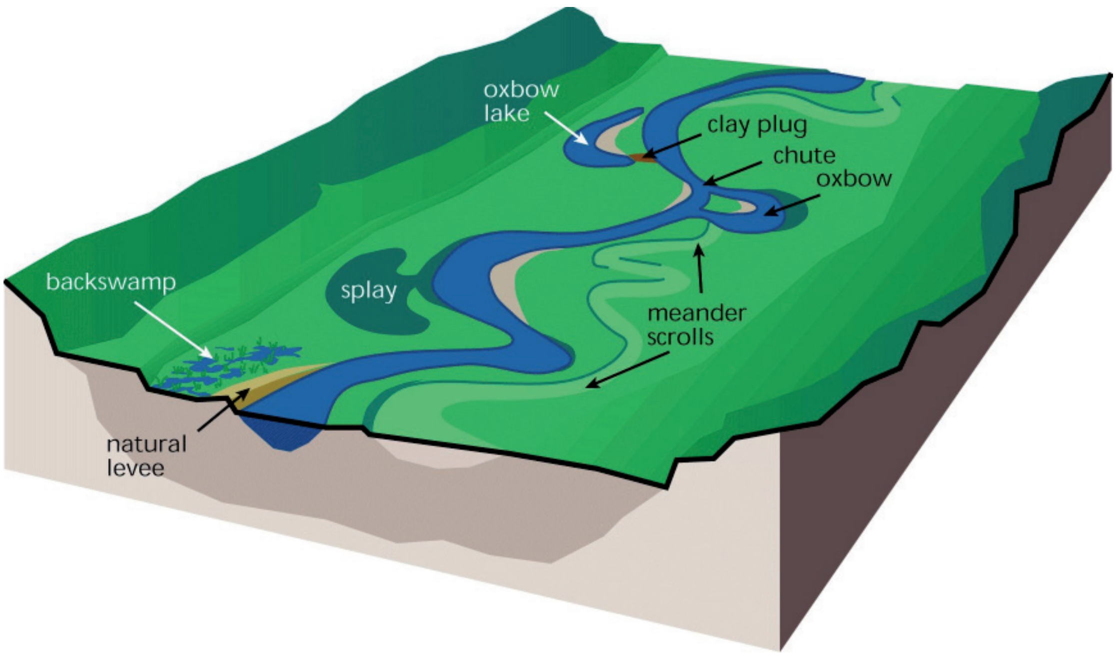


Figure II.4.46. The concept of a river corridor in the case of a meandering alluvial stream. The river corridor includes all of the landforms shown, as well as the river's flood plain (Federal Interagency Stream Restoration Working Group, 1998).

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.

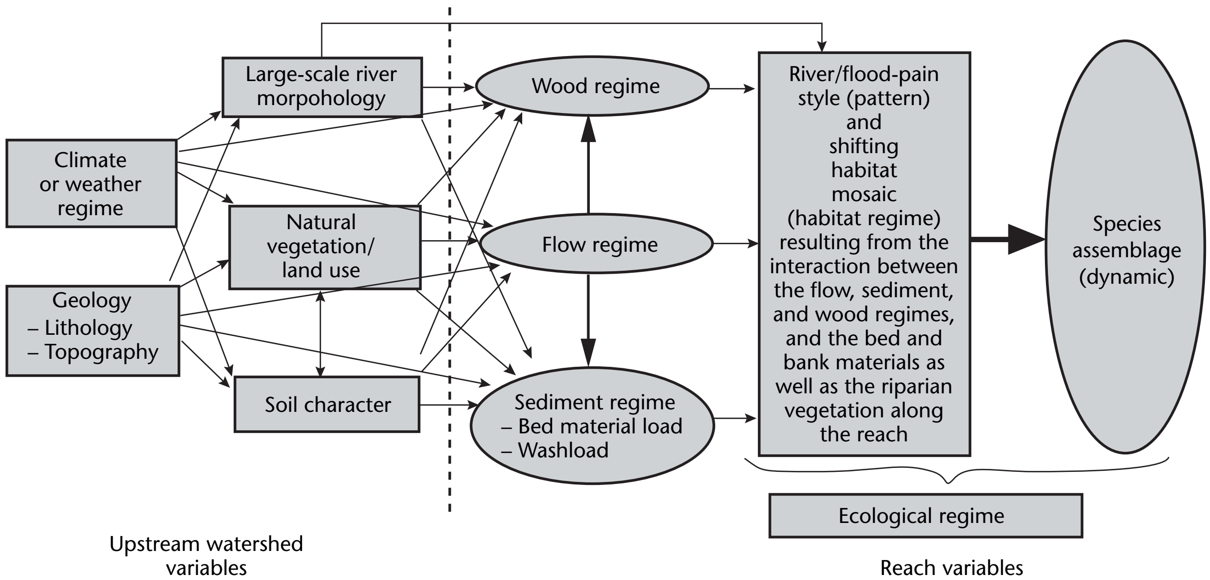


Figure II.4.47. Interaction among the catchment variables that determine the flow, sediment and wood regimes imposed on a reach from upstream. This, in turn, controls its morphological and habitat regimes – the river/flood-plain style and the shifting habitat mosaic, respectively – which together define 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

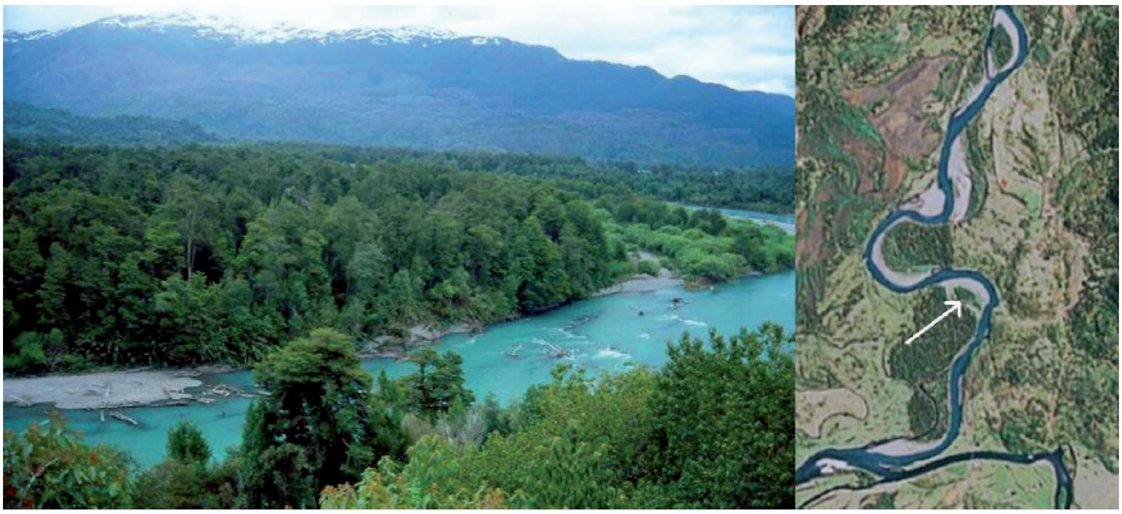


Figure II.4.48. Pristine river corridor of the Palena river in Chilean Patagonia, an active meandering single-thread sinuous system of high ecological integrity. Note the diversity of forms, water depths and velocities, ages of vegetation stands and the abundance of large woody debris in the channel. 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 hydrogeomorphologic 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.

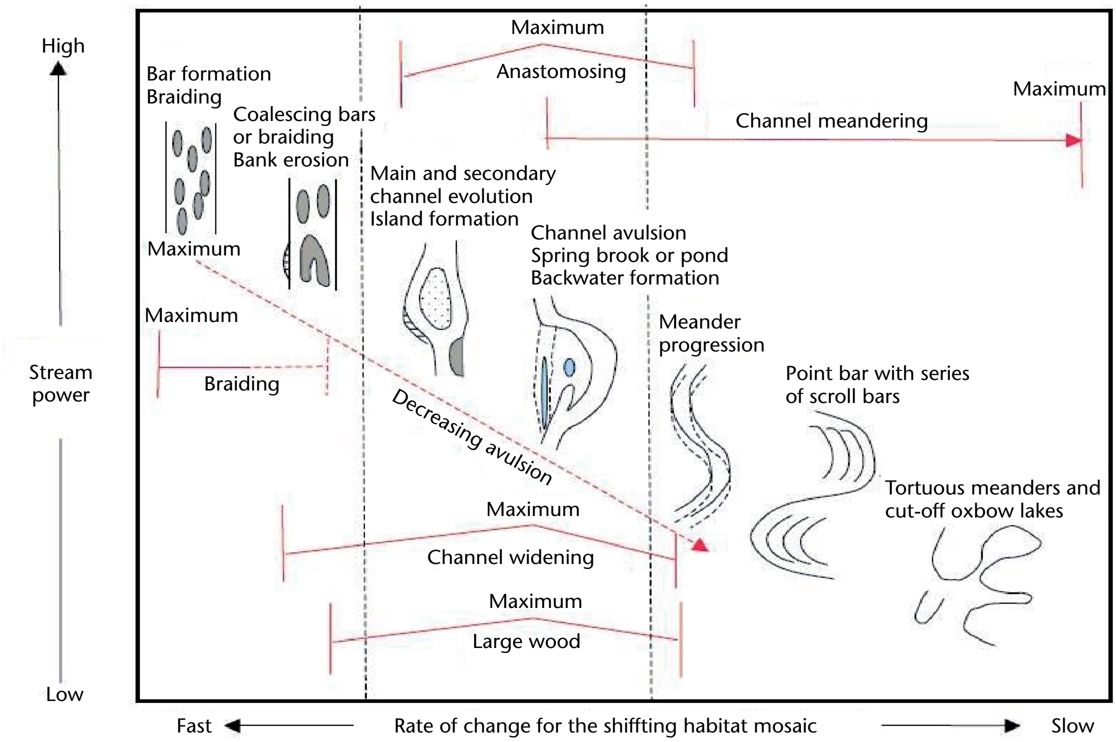


Figure II.4.49. Continuum of river styles associated with increasing stream power, indicating the varying rate of change for the shifting habitat mosaic, as well as some relevant hydrogeomorphic and biogeomorphic processes (simplified from Lorang and Hauer, 2006)

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 flood-plain 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

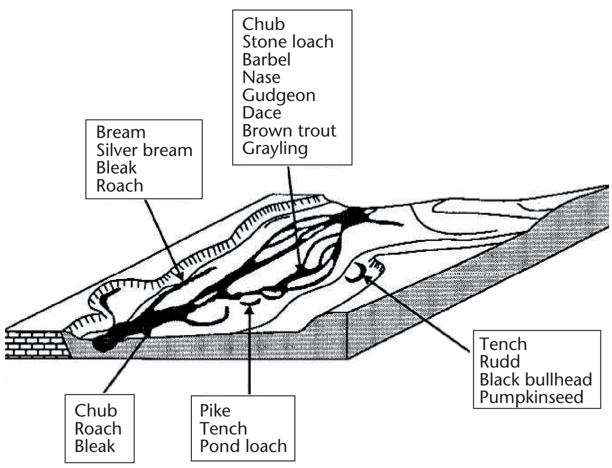


Figure II.4.50. Use of spawning habitat by different fish species in the wandering upper Rhone river, France (Roux and Copp, 1996)

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 population numbers, 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

Abramovitz, J.N., 1995: *Freshwater failures: The crises on five continents*. Washington, DC, World Watch 8, pp. 27–35.

Allan, J.D., 1995: Stream Ecology: Structure and Function of Running Waters. London, Chapman & Hall.

Allen G. Richard, Luis S. Pereira, Dirk Raes and Martin Smith, 1998: *Crop Evapotranspiration – Guidelines for Computing Crop Water Requirements*. FAO Irrigation and Drainage Paper 56. Rome, Food and Agriculture Organization of the United Nations.

Angermeier, P.L. and J.R. Karr, 1994: Biological integrity versus biological diversity as policy directives: Protecting biotic resources. *Bioscience*, 44(10):690–697.

Apmann, R.P., 1972: Flow processes in open channel bands. *Journal of the Hydraulics Division*. Proceedings of the American Society of Civil Engineers, 98:785–809.

ASCE, 1992: Design and Construction of Urban Stormwater Management Systems. Urban Water Resources Research Council of The American Society of Civil Engineers and the Water Environment Federation. 715 pp.

Ayres, R.S. and D.W. Westcot, 1985: *Water Quality for Agriculture*. FAO Irrigation and Drainage Paper 29.

Rome, Food and Agriculture Organization of the United Nations.
Basson, M.S., C. Triebel and J.. Van Rooyen, 1988: Analysis of a multi-baisn water resource system: A case study of the Vall River system. IWRA VIth World Congress on Water Rsources, Ottawa, Canada.
Basson, M.S., R.B. Allen, G.G.S Pegram and J.A. Van Rooyen, 1994: *Probabilistic Management of Water Resource and Hydropower Systems*. Highlands Ranch, Colorado,Water Resource Publications.
Bastiaanssen, W.G.M., 1998: Remote-sensing in water resources management: the state of the art. Colombo, International Water Management Institute, 118 pp.
Beschta, 1987: Conceptual models of sediment transport in streams. In: *Sediment Transport in Gravel-Bed Rivers* (C.R. Thorne, J.C. Bathurst and R.D. Hey, eds). John Wiley & Sons, pp. 396–397.
Boon, P.J., P. Calow and G.E. Petts, 1992: *River Conservation and Management*. Chichester, Wiley.
Box, G.E.P. and G.M. Jenkins, 1970: T*ime Series Analysis: Forecasting and Control*. San Francisco, Holden-Day.
Brookes, A., 1988: *Channelized Rivers: Perspectives for Environmental Management*. Chichester, Wiley.
Brookes, A. and F.D. Shields, 1996: *River Channel Restoration: Guiding Principles for Sustainable Projects*. Chichester, Wiley.
Calow, P. and G.E. Petts, 1994: *The Rivers Handbook*, Volumes 1 and 2. Oxford, Blackwell.
Campana, N. and C.E.M. Tucci, 1994: Estimativa de área impermeável de macro-bacias urbanas. *Revista Brasileira de Enghenharia – Caderno de Recursos Hídricos* 12:2, December.
Chin, D.A., 2006: *Water Quality Engineering in Natural Systems*. New York, Wiley–Interscience.
Chow, V.T. (ed.), 1964: *Handbook of Applied Hydrology*. New York, McGraw-Hill.
Chowdary, V.M., N.H. Rao and P.B.S. Sarma, 2003: GIS-based decision support system for ground-water assessment in large irrigation project areas. *Agricultural Water Management*, 62:229–252.
CIWEM (Chartered Institution of Water and Environmental Management), 1989: River Engineering – Part II, Structures and Coastal Defence Works. London.
Coussot, P., 1997: Mudflow Rheology and Dynamics, IAHR Monograph Series. Rotterdam, A.A. Balkema, 255 pp.
Cowx, I.G. and R.L. Welcomme (eds), 1998: *Rehabilitation of Rivers for Fish*. Oxford, Fishing News Books (Blackwell Science).
Cushing, C.E. and J.D. Allan, 2001: *Streams: Their Ecology and Life*. San Diego, California, Academic Press.
CWC (India), 1997: Draft guidelines for planning of conjunctive use in irrigation projects. Central Water Commission, Ministry of Water Resources, Government of India.
Danish Hydraulics Institute (DHI), 1990: MOUSE Modelling of Urban Sewer Systems on Microcomputers. Users Guide and Technical Reference. Edition 3.0.
Dastane, N.G., 1972: Effective Rainfall. FAO Irrigation and Drainage Paper 25. Rome, Food and Agriculture Organizaiton of the United Nations.
Desbordes, M and E. Servat, 1988: 'Towards a specific approach of urban hydrology in Africa' In: Hydrological Processes and Water Management in Urban Areas. Proceedings Duisburg Conf., Urban Water 88, April 231–237.
Doorenbos, J. and W.O. Pruitt, 1977: *Crop Water Requirements*. FAO Irrigation and Drainage Paper 24. Rome, Food and Agriculture Organization of the United Nations.
Doorenbos, J. and A.H. Kassam, 1979: *Yield Response to Water*. FAO Irrigation and Drainage Paper 33. Rome, Food and Agriculture Organization of the United Nations.
Dunbar, M.J. and M.C. Acreman, 2001: Applied hydro-ecological science for the twenty-first century. In: *Hydro-Ecology: Linking Hydrology and Aquatic Ecology* (M.C. Acreman, ed.). IAHS Publication No. 266. Wallingford, IAHS Press, pp. 1–17.
Engelund, F. and E. Hansen, 1967: A Monograph on Sediment Transport in Alluvial Streams. Denmark, Teknisk Forlag.
Engman, E.T. and R.J. Gurney, 1991: *Remote-sensing in Hydrology*. London, Chapman and Hall, 225 pp.
Federal Interagency Stream Restoration Working Group (FISRWG), 1998: *Stream Corridor Restoration: Principles, Processes, and Practices*. GPO Item No. 0120-A; SuDocs No. A 57.6/2:EN 3/PT.653.
Feldman, A.D., 1995: HEC-1 flood hydrograph package. In: *Computer Models of Watershed Hydrology* (Vijay P. Singh, ed.). Fort Collins, Colorado, Water Resources Publications.
Framji, K.K. (ed.), 1987: Improvement in irrigation management with special reference to developing countries. In: *State-of-the-Art: Irrigation, Drainage and Flood Control*, No. 4. New Delhi, International Commission on Irrigation and Drainage.
Framji, K.K. and B.C. Garg, 1978: *Flood Control in the World – A Global Review*, Volume II. International Commission on Irrigation and Drainage.
Garde, R.J. and K.G. Ranga Raju, 2000: *Mechanics of Sediment Transportation and Alluvial Stream Problems*. New Delhi, New Age International (P) Ltd. Publications, 696 pp.
Gardiner, J.L. (ed.), 1991: *River Projects and Conservation: A Manual for Holistic Appraisal*. Chichester, Wiley.
Gore, J.A. and G.E. Petts (eds), 1989: *Alternatives in Regulated River Management*. Boca Ratonne, Florida, CRC Press.

Graf, W.H., 1984: *Hydraulics of Sediment Transport*. Littleton, Colorado, Water Resources Publications, 513 pp.

Guy, H.P., 1970: Fluvial Sediment Concepts. Chapter C1, Book 3, Techniques of Water Resources Investigations. Washington, DC, United States Geological Survey.

Hajilal, M.S., N.H. Rao and P.B.S. Sarma, 1998: Planning intra-seasonal irrigation requirements in large projects. *Agricultural Water Management*, 37(2):163–182.

Harper, D.M., 1991: *Eutrophication of Freshwaters: Principles, Problems, and Restoration*. London, Chapman-Hall.

Harper, D.M. and A.J.D. Ferguson, 1995: *The Ecological Basis for River Management*. Chichester, Wiley.

Heaney, J.P, D. Sample and L. Wright, 2002: Costs of urban stormwater control. USEPA Report EPA/600/R-02/021, January 2002. Cincinnati, Ohio, United States Environmental Protection Agency.

Henderson-Sellers, B. and H.R. Markland, 1987: *Decaying Lakes: The Origins and Control of Cultural Eutrophication*. Chichester, Wiley.

Hipel, W.H., A.I. McLeod and W.C. Lennox, 1977: Advances in Box–Jenkins Modelling. *Water Resources Research*, 13(3):567–575, http://fisher.stats.uwo.ca/faculty/aim/vita/pdf/Advances1.pdf.

Hughes, D.A. and P. Hannart, 2003: A desktop model used to provide an initial estimate of the ecological instream flow requirements of rivers in South Africa. *Journal of Hydrology*, 270(3–4):167–181.

Hydrologic Engineering Center (HEC), 1977: *Storage, Treatment, Overflow, Runoff Model STORM: User's manual, Generalized Computer Program 723-S8-l7520*. Davis, California, Hydrologic Engineering Center, Corps of Engineers.

Hynes, H.B., 1972: *The Ecology of Running Waters*. Toronto, University of Toronto Press.

Intergovernmental Panel on Climate Change (IPCC), 2001: *Climate Change 2001: The Scientific Basis*. New York: Cambridge University Press.

———, 2007: *Fourth Assessment Report – Climate Change 2007*. Geneva.

International Atomic Energy Agency (IAEA), 1981: *Ultimate Heat Sink and Directly Associated Geat Transport Systems for Nuclear Power Plants: A Safety Guide*. Safety Series No. 50-SG-D6. Vienna.

International Commission on Irrigation and Drainage (ICID), 1996: *Manual on Non-structural Approaches to Flood Management*. Delhi.

Jeffries, M.J. and D.H. Mills, 1995: *Freshwater Ecology: Principles and Applications*. Chichester, Wiley.

Jowett, I.G., 1997: Instream flow methods: A comparison of approaches. *Regulated Rivers: Research and Management*, 13:115–127.

Karr, J.R., 1996: Ecological integrity and ecological health are not the same. In: *Engineering Within Ecological Constraints* (P.C. Schulze, ed.). Washington, DC,National Academy of Engineering, pp. 97–109.

Karr, J.R. and E.W. Chu, 1999: *Restoring Life in Running Waters*. Washington, DC, Island Press.

Kurtas W.P. and J.M. Norman, 1996: Use of remote-sensing for evapotranspiration monitoring over land surfaces. *Journal of Hydrological Sciences*, 41:495–516.

Lawrence, I., 2001: *Integrated urban land and water management – Planning and design guidelines*. Cooperative Research Centre for Freshwater Ecology, University of Canberra, ACT 2601. Technical Report 1/2001. February 2001. Australia.

Leopold, L.B., 1994: *A View of the River*. Cambridge, Massachusetts, Harvard University Press.

Lorang, M.S and F.R. Hauer, 2006: Fluvial geomorphic processes. In: *Methods in Stream Ecology* (F.R. Hauer and G.A. Lamberti, eds). Second edition, Chapter 7. San Diego, Academic Press/Elsevier.

Lorang, M.S., D.C. Whited, F.R. Hauer, J.S. Kimball and J.A. Stanford, 2005: Using airborne multispectral imagery to evaluate geomorphic work across flood plains of gravel-bed rivers. *Ecological Applications*, 15(4):1209–1222.

Loucks, D.P., J.R. Stedinger and D.A. Haith, 1981: *Water Resources Systems Planning and Analysis*. Prentice-Hall.

Maidment, D.R. (ed.), 1992: *Handbook of Hydrology*. New York, McGraw Hill.

Maksimovic, C. and M. Radojkovic, 1986: *Urban Drainage Catchments – Selected Worldwide Rainfall-Runoff Data from Experimental Catchments*. Oxford, Pergamon Press.

Meier, C.I. 1998. The ecological basis of river restoration: 1. River ecology for hydraulic engineers. 2. Defining restoration from an ecological perspective. Proceedings of the Conference Engineering Approaches to Ecosystem Restoration, Denver, Colorado. Reston, Virginia, American Society of Civil Engineers.

Morisawa, M.E., 1985: *Rivers: Form and Process*. Geomorphology Texts, No. 7. London, Longman.

Munich Re, 2006: *Annual Review: Natural Catastrophes 2005*, Knowledge series, Topics Geo. Munich. http://www.munichre.com/publications/302-04772_en.pdf.

Musiake, K., T. Oki, T. Nakaegawa and K. Wakasa, 1995: Verification experiment of extraction of soil moisture information using SAR mounted on JERS-1 and ERS-1, Final report of JERS-1/ERS-1 Systems Verification Programme–Ministry of International Trade and Industry, Kyoto, pp. 617–624.

National Research Council, 1973: Proceedings of the Ninth Hydrology Symposium on Fluvial Processes and Sedimentation, 8–9 May 1973, Edmonton. Ottawa, Ontario, Inland Waters Directorate, Department of the Environment.

———, 1992: *Restoration of Aquatic Ecosystems*. Washington, DC, National Academy Press.

Negev, M., 1972: Suspended Sediment Discharge in Western Watersheds of Israel. Hydrological Paper 14. Jerusalem, Hydrological Service.

Neill, C.R., 1973: *Guide to Bridge Hydraulics*. Roads and Transportation Association of Canada. Toronto, University of Toronto Press.

Nuttle, W.K., 2002: Is ecohydrology one idea or many? *Hydrological Sciences Journal*, 47:805-807.

Odum, E. and G.W. Barrett, 2004: *Fundamentals of Ecology*. Fifth edition. Brooks Cole.

Pegram, G.G.S. and R.S. McKenzie, 1991: Synthetic streamflow generation in the Vaal River system study. Proceedings of the South African Institution of Civil Engineering, January, pp. 15–24.

Permanent International Association of Navigation Congress (PIANC), 1987: Guidelines for the design and construction of flexible revetments incorporating geotextiles for inland waterways, Report of Working Group 4 to Permanent Technical Committee 1. Brussels.

Petts, G.E., 1984: *Impounded Rivers: Perspectives for Ecological Management*. Chichester, Wiley.

Petts, G.E. and C. Amoros (eds), 1996: *Fluvial Hydrosystems*. London: Chapman and Hall.

Prihar, S.S., K.L. Khera, K.S Sandhu and B.S. Sandhu, 1976: Comparison of irrigation schedules based on pan evaporation and growth stages in winter wheat, *Agronomy Journal*, 68:650–653.

Prosser, M.J., 1986: Propeller-Induced Scour, Report RR2570, British Hydromechanics Research Association. Bedford, Cranfield.

Rao, N.H., P.B.S. Sarma and Subhash Chander, 1988*a*: A simple dated water-production function for use in irrigated agriculture. *Agricultural Water Management*, 13(1):25–32.

Rao, N.H., P.B.S. Sarma and Subhash Chander, 1988*b*: Irrigation scheduling under a limited water supply. *Agricultural Water Management*, 15(2):165–175.

Rao, N.H. and D.H. Rees, 1992: Irrigation scheduling of rice with a crop growth simulation model. *Agricultural Systems*, 39(2):115–132.

Raudkivi A.J., 1998: *Loose Boundary Hydraulics*. Rotterdam, A.A. Balkema, 496 pp.

Rickenmann D., 1991: Hyperconcentrated flow and sediment transport at steep slopes. *Journal of Hydraulic Engineering*, 117(11):1419–1439.

Roux, A.L. and G.H. Copp. 1996. Fish populations in rivers. In: *Fluvial Systems* (G.E. Petts and C. Amoros, eds). Chapter 8, pp. 167–183. London: Chapman & Hall.

Ryding, S.O. and W. Rast (eds), 1989: *The Control of Eutrophication of Lakes and Reservoirs*. Man and the Biosphere Series. Paris: UNESCO.

Santos, R.J.L., 1998: Incerteza na estimativa do hidrograma de projeto com base em modelos hidrológicos. MSc thesis. Porto Alegre, Institute of Hydraulic Research.

Schulze, R.E. and L.A. Perks, 2000: Assessment of the Impact of Climate Change on Hydrology and Water Resources in South Africa. ACRU Report No. 33, School of Bioresources Engineering and Environmental Hydrology, University of Natal, South Africa.

Schumm, S.A., 2005: *River Variability and Complexity*. Cambridge, Cambridge University Press.

SCS, 1975: Urban Hydrology for Small Watersheds. Technical Release 55. Washington, United States Department of Agriculture.

Simons, D.B. and E.V. Richardson, 1961: Forms of bed roughness in alluvial channel. *Journal of the Hydraulics Division*, 87(3):801–105.

Singh, B., 1989: *Fundamentals of Irrigation Engineering*, Roorkee, Nem Chand & Bros.

Smart, G.M., 1984: Sediment transport formula for steep channels. *Journal of Hydraulic Engineering*, (110)3:267–276.

Stanford, J.A., J.V. Ward, W.J. Liss, C.A. Frissel, R.N. Williams, J.A. Lichatowich and C.C. Coutant, 1996: A general protocol for restoration of regulated rivers. *Regulated Rivers: Research and Management*, 12(4–5):391–413.

Stringham, G.E., 1988: Surge Flow Irrigation. Utah Agricultural Experiment Station Research Bulletin, 515 pp.

Stringham, G.E. and J. Keller, 1979: Surge flow for automatic irrigation. ASCE Irrigation and Drainage Division Speciality Conference, Albuquerque, New Mexico, pp. 131–142.

Svanidze, G.G., 1977: Mathematicheskoe modelirovanie gidrologicheskikh ryadov (Mathematical modelling of hydrological series). Leningrad, Gidrometeoizdat. Also published in English: 1980, Water Resources Publications LLC.

Szöllösi-Nagy, A. and C. Zevenbergen (eds), 2005: *Urban Flood Management*. London: A.A. Balkema Publishers.

Tharme, R.E., 2003: A global perspective on environmental flow assessment: emerging trends in the development and application of environmental flow methodologies for rivers. *River Research and Applications*, 19(5–6):397–441.

Tharme, R.E. and J.M. King, 1998. Development of the building block methodology for instream flow assessments, and supporting research on the effects of different magnitude flows on riverine ecosystems. Water Research Commission Report No. 576/1/98, South Africa.

Thomann, R.V. and J.A. Mueller, 1987: *Principles of Surface Water Quality Modeling and Control*. Harper Collins Publishers.

Thorne C. R., J.C. Bathurst and R.D. Hey, 1987: *Sediment Transport in Gravel-Bed Rivers*. John Wiley and Sons.

Tucci, C.E.M. (ed.), 2001: *Urban Drainage in Humid Tropics*. Volume 1 of *Urban Drainage in Specific Climates* (C. Maksimovic, ed.). IHP-V Technical

Documents in Hydrology No. 40, . Paris, UNESCO. http://unesdoc.unesco.org/images/0012/001228/122848eo.pdf.

United Nations, 1977: Mar del Plata Action Plan. United Nations Water Conference, Argentina.

United Nations 1987: *Our Common Future*. World Commission on Environment and Development. Oxford, Oxford University Press.

United Nations, 1992: International Conference on Water and the Environment: Development Issues for the Twenty-first Century, 26–31 January 1992, Dublin, Ireland.

United Nations, 2000: Millennium Development Goals, Millennium Declaration. New York.

United Nations Educational, Scientific and Cultural Organization/World Meteorological Organization (UNESCO/WMO), 1992: *International Glossary of Hydrology*. Third edition.

———, 1997: *Water Resources Assessment Activities: Handbook for Review of National Capabilities*.

Urbonas, B. and P. Stahre, 1993: *Stormwater Best Management Practices and Detention*. Englewood Cliffs, New Jersey, Prentice Hall.

US Army Corps of Engineers (USACE), 1995: Flood Proofing Regulations. https://www.nwo.usace.army.mil/nfpc/fpreg/ace6.htm.

Van Rijn, L.C., 1984: Sediment transport, part III: Bed forms and alluvial roughness. J*ournal of Hydraulic Engineering*, 110(12):1732–1754.

Vanoni, V.A. (ed.), 1975: *Sedimentation Engineering*. ASCE Manual 54. New York, American Society of Civil Engineers.

Vidal, A. and J.A. Sagardoy (eds), 1995: *Use of Remote-Sensing Techniques in Irrigation and Drainage*. Water Reports 4. Proceedings of the Expert Consultation, Montpellier, France, 2–4 November 1993. Rome, Food and Agriculture Organization of the United Nations, 202 pp.

World Commission on Dams (WCD), 2000: Dams and Development. A New Framework for Decision-making. The report of the World Commission on Dams. London, Earthscan Publications.

World Health Organization, 1988: *Urbanization and its Implications for Child Health: Potential for Action*. Geneva.

World Meteorological Organization, 1981: *Meteorological and Hydrological Aspects of Siting and Operation of Nuclear Power Plants*. Volume II – Hydrological Aspects. Technical Note No. 170, WMO-No. 550, Geneva.

———, 1989: (Long, Yuqian), *Manual on Operational Methods for the Measurement of Sediment Transport*. WMO-No. 686, Geneva.

———, 1992: *Hydrological Aspects of Accidental Pollution of Water Bodies*, Operational Hydrology Report No. 37, WMO No. 754, Geneva.

———, 1999: *Contaminants in Rivers and Streams – Prediction of Travel Time and lLongitudinal Dispersion*. Operational Hydrology Report No. 45, WMO-No. 886, Geneva.

———, 2003: *Manual on Sediment Management and Measurement*. Operational Hydrology Report No. 47, WMO-No. 948, Geneva.

———, APFM, 2004: *Integrated Flood Management Concept Paper*, APFM Technical Document No. 1, second edition. Associated Programme on Flood Management, Geneva.

———, APFM, 2006*a*: *Social Aspects and Stakeholder Involvement in Integrated Flood Management*, APFM Technical Document No. 4, Flood Management Policy Series. Associated Programme on Flood Management, Geneva.

———, APFM, 2006*b*: *Legal and Institutional Aspects of Integrated Flood Management*, APFM Technical Document No. 2, Flood Management Policy Series. Associated Programme on Flood Management, Geneva.

———/GWP, 2006: *Environmental Aspects of Integrated Flood Management*. APFM Technical Document No. 3, Flood Management Policy Series. World Meteorological Organization/Global Water Partnership, Geneva.

———/United Nations Educational, Scientific and Cultural Organization, 1991: Progress in the implementation of the Mar del Plata Action Plan and a strategy for the 1990s. Report on Water Resources Assessment.

Wright-McLaughlin Engineers, 1969: *Urban Storm Drainage Criteria Manual*. Two volumes. Denver, Colorado.

Yalin M.S., 1992: *River Mechanics*. Oxford, Pergamon Press, 220 pp.

Yalin, M.S. and A.M. Ferreria da Silva, 2001: *Fluvial Processes*. Monograph. Delft, International Association of Hydraulic Engineering and Research.

Yen, B.C., 1986: Rainfall-runoff process on urban catchments and its modelling. In: *Proceedings of the International Symposium on Comparison of Urban Drainage Models with Real Catchments Data*, 9–11 April 1986, Dubrovnik (C. Maksimovic and Radojkovic, eds).

Zalewski, M., 2000: Ecohydrology: The scientific background to use ecosystem properties as management tools toward sustainability of water resources. *Ecological Engineering*, 16:1–8.

Zhaohui Wan and Zhaoyin Wang, 1994: Hyperconcentrated Flow, IAHR Monograph Series, Rotterdam, A.A. Balkema, 290 pp.

CHAPTER 5

EXTREME VALUE ANALYSIS

5.1 INTRODUCTION

The purpose of frequency analysis is to analyse past records of hydrologic variables so as to estimate future occurrence probabilities. The data used in the analysis must be evaluated in terms of the objectives, length of records available and completeness of records. It must also satisfy certain statistical criteria such as randomness, independence, homogeneity and stationarity. A frequency analysis can be performed using single-site data, regional data or both. It can also include historical information and reflect physical constraints.

Because hydrological phenomena are characterized by great variability, randomness and uncertainty, it should, therefore be recognized that statistical analysis of hydrological data will not always yield a true answer. The sources of uncertainty in frequency analysis include representativness of the analytical approach, selection of the probability distribution and estimation of parameters.

Hydrological analysis is generally based on well-established principles of hydrodynamics, thermodynamics and statistics. However, the central problem in hydrological analysis is the application of these principles in a natural environment that is non-homogeneous, sparsely sampled and only partially understood. The events sampled are usually unplanned and uncontrolled. Analyses are performed to obtain spatial and temporal information about hydrological variables, regional generalizations and relationships among the variables. Analyses can be performed using deterministic, parametric, probabilistic and stochastic methods. An analysis based on the deterministic approach follows the laws that describe physical and chemical processes. In the parametric approach, an analysis is performed by intercomparison of hydrological data recorded at different locations and times. In the probabilistic approach, the frequency of occurrence of different magnitudes of hydrological variables is analysed. In the stochastic approach, both the sequential order and the frequency of occurrence of different magnitudes are analysed often using time-series methods. Evidence continues to accumulate documenting the dynamic and nonlinear character of the hydrological cycle. In the case of extreme events, our major interest is not in what has occurred, but the likelihood that further extreme and damaging events will occur at some point in the future.

The occurrence of many extreme events in hydrology cannot be forecasted on the basis of deterministic information with sufficient skill and lead time. In such cases, a probabilistic approach is required to incorporate the effects of such phenomena into decisions. If the occurrences can be assumed to be independent in time, in that the timing and magnitude of an event bears no relation to preceding events, then frequency analysis can be used to describe the likelihood of any one or a combination of events over the time horizon of a decision. Hydrological phenomena commonly described by frequency analysis include storm precipitation (5.7), low flows (5.8) and annual flood maxima (5.9).

Both the detail and precision of the analysis should be consistent with the quality and sampling adequacy of the available data and with the accuracy required by the application of the analysis. Consideration should be given to the relationship between the cost and time devoted to an analysis and to the benefits expected. Traditionally, graphical and very simple computational methods have proven more cost effective than more sophisticated methods, and they may be sufficiently accurate for the data and purposes involved. However, the widespread availability of personal computing equipment, with general-purpose statistical software and computation environments such as spreadsheets, has largely replaced hand computational procedures. A major advantage of the modern computational environment is that it should improve an agency's ability to store, retrieve and analyse data. Further, the graphical capabilities of personal computers should greatly enhance the ability of hydrologists to review and understand their data, as well as the results and the computations that they perform.

5.2 STATISTICAL SERIES AND RETURN PERIODS [HOMS H83]

In frequency analysis, a series is a convenient sequence of data, such as hourly, daily, seasonal or annual observations of a hydrological variable. If

the record of these observations contains all the events that occurred within a given period, the series is called a complete duration series. For convenience, the record often contains only events of magnitude above a pre-selected base or threshold level; this is called a partial duration series or peaks-over-threshold series. A series that contains only the event with the largest magnitude that occurred in each year is called an annual maximum series.

The use of the annual maximum series is very common in frequency analyses for two reasons. The first is for convenience, as most data are processed in such a way that the annual series is readily available. The second is that there is a simple theoretical basis for extrapolating the frequency of annual series data beyond the range of observation. With partial series data, such theory is not as simple because one must consider the arrival process of floods within a year and the distribution of the magnitude of floods when they do occur. Another problem with partial duration series is the lack of independence of events that might follow one another in close sequence, as well as seasonal effects. However, if the arrival rate for peaks over the threshold is large enough and can be modelled by simple two-parameter distributions, for example 1.65 for the Poisson arrival with exponential exceedances model, it should yield more accurate estimates of flood quantiles than the corresponding annual flood frequency analyses. However, when fitting a three-parameter distribution, such as the generalized Pareto distribution for exceedances with Poisson arrivals, there appears to be no advantage in using a partial duration series no matter how many floods are recorded on average each year (Martins and Stedinger, 2000). It should not be a surprise that recording the value of a great many small events does not tell us much about the risk of very large events occurring unless the structure of the model is fairly rigid.

A limitation of annual series data is that each year is represented by only one event. The second highest event in a particular year may be higher than the highest in some other years, yet it would not be contained in the series. The use of partial duration series can address this issue because all peaks above the specified threshold are considered.

The complete duration series may be required for the stochastic approach in which independence is not required. It may also serve for a probabilistic analysis of data from arid regions where the events are rare and almost independent.

The return period T of a given level is the average number of years within which the event is expected to be equalled or exceeded only once. The return period is equal to the reciprocal of the probability of exceedance in a single year. If the annual exceedance probability is denoted $1/T_a$, where T_a is the annual return period, the relationship between the annual return period and the return period in the partial duration series can be expressed as follows:

$$1/T_a = 1 - exp\{-\lambda\, q_e\} = 1 - exp\{-l/T_p\} \tag{5.1}$$

where $T_p = 1/(\lambda\ qe)$ is the average return period in the partial duration series with λ being the arrival rate for peaks over the threshold and q_e is the probability that when such a flood occurs, it exceeds the level of concern. This equation can be solved for T_p to obtain:

$$T_p = 1 / ln\ [1 - 1/T_a] \tag{5.2}$$

T_p is less than T_a because more than one event can occur per year in a partial duration series. For return periods exceeding ten years, the differences in return periods obtained with the annual and partial series is inconsequential. Table II.5.1 compares the return periods for an annual maximum series and a partial duration series. This formula is based on the assumption that floods in the partial duration series occur independently in time and at a constant rate; relaxation of that assumption yields different relationships (Robson and Reed, 1999). NERC (1975) observes that the actual probabilistic model for arrivals with regard to large return period events is not particularly important, provided that different models yield the same average number of arrivals per year (see also Cunnane, 1989).

Table II.5.1. Corresponding return periods for annual and partial series

Partial series	*Annual series*
0.50	1.16
1.00	1.58
1.45	2.00
2.00	2.54
5.00	5.52
10.00	10.50

5.3 PROBABILITY DISTRIBUTIONS USED IN HYDROLOGY [HOMS H83, X00]

Probability distributions are used in a wide variety of hydrological studies, including studies of extreme high and low flows, droughts, reservoir volumes, rainfall quantities and in time-series models. Table II.5.2 lists the most commonly used distributions in hydrology. Their mathematical definitions are given in a number of references (Kite, 1988; Cunnane, 1989; Bobee and Ashkar, 1991; Stedinger and others, 1993; Clark, 1994; Kottegoda and Rosso, 1997 and Hosking and Wallis, 1997).

Numerous probability distributions have been introduced in the literature to model hydrological phenomena such as extreme events. Despite intensive research and study, no particular model is considered superior for all practical applications. The user should, therefore, screen available models in the light of the problem to be solved and the nature of the available data. Consequently, only some distributions that are in common use are reviewed in this chapter. The contending distributions that fit the observed data satisfactorily usually differ significantly in the tail of the distribution, especially when extrapolation is involved. No general guidance is available for extrapolating distributions, particularly beyond twice the available record length. The decision regarding which distribution to use should be based on the comparison of the suitability of several candidate distributions. The advantages and disadvantages of the various methods that can be used for this objective are discussed in 5.6.

Annual totals, such as flow volumes or rainfall depths, tend to be normally distributed or almost so because of the forces described by the central limit theorem of statistics. Monthly and weekly totals are less symmetric, displaying a definite skewness that is mostly positive and cannot usually be modelled by the normal distribution. Annual extremes – high or low – and peaks over a threshold tend to have skewed distributions. The part of a sample that lies near the mean of the distribution can often be described well by a variety of distributions. However, the individual distributions can differ significantly and very noticeably from one another in the values estimated for large return periods, as well as very small cumulative probabilities. As hydraulic design is often based on estimates of large recurrence-interval events, it is important to be able to determine them as accurately as possible. Hence, the selection of the distribution is very important for such cases. The choice of distributions is discussed in the references cited above, which include discussions on the methods available for choosing between distributions. This is also discussed in 5.6.

Generally, mathematical distributions having three parameters, such as those appearing in Table II.5.2, are selected so as to make the distribution matches the available data more consistently. In some cases an empirical distribution can be used to describe the data, thereby avoiding the use of mathematical parametric distributions.

Use of a mathematical distribution has several advantages:

(a) It presents a smooth and consistent interpretation of the empirical distribution. As a result, quantiles and other statistics computed using the fitted distribution should be more accurate than those computed with the empirical distribution;
(b) It provides a compact and easy-to-use representation of the data;
(c) It is likely to provide a more realistic description of the range and likelihood of values that the random variable may assume. For example, by using the empirical distribution, it is implicitly assumed that no values larger or smaller than the sample maximum or minimum can occur. For most situations this is entirely unreasonable.

There are several fundamental issues that arise in selecting a distribution for frequency analysis (Stedinger and others, 1993):

(a) What is the true distribution from which the observations are drawn?
(b) Is a proposed flood distribution consistent with available data for a particular site?
(c) What distribution should be used to obtain reasonably accurate and robust estimates of flood quantiles and flood risk for hydrological design purposes?

Unfortunately, the answer to the first question will never be known, and it might not be much help if it were. The true distribution of the data could be incredibly complex with more parameters than a hydrologist could ever hope to estimate. Thus, the aim is to establish a good, but simple approximation of the true distribution of the events. Standard goodness-of-fit statistics and probability plots can, at least in part, address the second question, as they will sometimes show that particular distributions are not consistent with the available data. There may be pragmatic considerations to prevent the use of a distribution for a particular sample. For example,

the distribution may be upper bounded at what is considered to be an unreliable low value, thereby not providing an acceptable estimate of extreme conditions. As a practical matter, many national agencies look at the problem from the point of view of the third question: What distribution coupled with a reasonable fitting procedure will yield good estimates of risk in their region of the world? Thus, the aim is not to seek absolute truths. Instead, the goal is to develop practical procedures which, with the data in hand or data that can be collected, will provide a good approximation of the frequency relationships of interest. Over the past four decades, various distributions have been introduced for use in hydrological frequency analysis. The following section provides an overview of some of these distributions.

5.3.1 Normal family: N, LN and LN3

5.3.1.1 Normal distribution

The normal distribution (N) is useful in hydrology for describing well-behaved phenomena, such as the total annual flow. The probability density function for a normal random variable X is given in Table II.5.2, and it is unbounded both above and below, with mean μ_x and variance σ^2_x. The normal distribution's skewness coefficient is zero, because the distribution is symmetric. The cumulative distribution function (CDF) of the normal distribution is not available in closed form, but books on statistics include tables of the standardized normal variate z_p. The quantity z_p is an example of a frequency factor because the pth quantile x_p of any normal distribution with mean μ and variance σ^2 can be written as follows:

$$x_p = \mu + \sigma\, z_p \tag{5.3}$$

5.3.1.2 Log-normal distribution

In general, flood distributions are positively skewed and not properly described by a normal distribution. In many cases the random variable corresponding to the logarithm of the flood flows will be adequately described by a normal distribution. The resulting two-parameter log-normal (LN) distribution has the probability-density function given in Table II.5.2. Often, the logarithms of a random variable X are not distributed normally. In such cases, introducing a boundary parameter ζ before taking logarithms can solve this problem, yielding a three-parameter log-normal distribution (LN3) (Stedinger and others, 1993) so that:

$$Y = ln\, [X - \zeta] \tag{5.4}$$

would have a normal distribution. Thus:

$$X = \zeta + \exp\,(Y) \tag{5.5}$$

has a LN3 distribution. In terms of the frequency factors of the standard normal distribution z_p, the quantiles of a log-normal distribution are as follows:

$$x_p = \zeta + \exp\,(\mu_Y + \sigma_Y\, z_p) \tag{5.6}$$

where μ_Y and σ_Y are the mean and standard deviation of Y. Parameter estimation procedures are compared by Stedinger (1980).

5.3.2 Extreme value distributions: Gumbel, generalized extreme value and Weibull

Gumbel (1958) defined three types of extreme value distributions which should describe the distribution of the largest or smallest value in a large sample. They have been widely used in hydrology to describe the largest flood or the lowest flow.

5.3.2.1 Gumbel distribution

Annual floods correspond to the maximum of all of the flood flows that occur within a year. This suggests their distribution is likely to be a member of a general class of extreme value (EV) distributions developed in Gumbel (1958). Let $X_1,\ldots,X_n$ be a set of annual maximum discharges and let $X = max\{X_i\}$. If the X_i are independent and identically distributed random variables unbounded above, with an exponential-like upper tail, then for large n the variate X has an extreme value (EV) type I distribution or Gumbel distribution with cumulative distribution function given in Table II.5.2.

Landwehr and others (1979) and Clarke (1994) discuss estimation procedures and Hosking (1990) has shown that L–moments provide accurate quantile estimates for the small sample sizes typically available in hydrology.

5.3.2.2 Generalized extreme value distribution

The generalized extreme value distribution spans the three types of extreme value distributions for maxima. The Gumbel and generalized extreme value distribution distributions are widely used for flood frequency analyses around the world (Cunnane, 1989). Table II.5.2 provides the cumulative distribution function of the generalized extreme value distribution.

Table II.5.2. Commonly used frequency distributions (after Stedinger and others, 1993)

Distribution	*Probability density function and/or cumulative distribution function*	*Range*	*Moments*
Normal	$f_X(x) = \frac{1}{\sqrt{2\pi\sigma_X^2}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu_X}{\sigma_X}\right)^2\right]$	$-\infty < x < \infty$	μ_X and σ^2_X, $\gamma_X = 0$
Log-normal[a]	$f_X(x) = \frac{1}{x\sqrt{2\pi\sigma_Y^2}} \exp\left[-\frac{1}{2}\left(\frac{\ln(x)-\mu_Y}{\sigma_Y}\right)^2\right]$	$0 < x$	$\mu_X = \exp[\mu_Y + \sigma^2_Y/2]$ $\sigma^2_X = \mu^2_X \{\exp[\sigma^2_Y] - 1\}$ $\gamma_X = 3CV_X + CV^3_X$
Pearson type III	$f_X(x) = \lvert\beta\rvert [\beta(x-\xi)]^{\alpha-1} \exp[-\beta(x-\xi)]/\Gamma(\alpha)$ (for $0 < \beta$ and $\xi = 0$: $\gamma_X = 2(CV_X)$)	$0 < \alpha$ for $0 < \beta$: $\xi < x$ for $\beta < 0$: $x < \xi$	$\mu_X = \xi + \alpha/\beta$; $\sigma^2_X = \alpha/\beta^2$ and $\gamma_X = 2/\sqrt{\alpha}$ and $\gamma_X = -2/\sqrt{\alpha}$
Log-Pearson type III	$f_X(x) = \lvert\beta\rvert \{\beta[\ln(x) - \xi]\}^{\alpha-1} \exp\{-\beta[\ln(x) - \xi]\}/x\Gamma(\alpha)$ for $\beta < 0$, $0 < x < \exp(\xi)$; for $0 < \beta$, $\exp(\xi) < x < \infty$	See Stedinger and others (1993).	
Exponential	$f_X(x) = \beta \exp\{-\beta(x-\xi)\}$ $F_X(x) = 1 - \exp\{-\beta(x-\xi)\}$	$\xi < x$ for $0 < \beta$	$\mu_X = \xi + 1/\beta$; $\sigma^2_X = 1/\beta^2$ $\gamma_X = 2$
Gumbel	$f_X(x) = (1/\alpha) \exp\{-(x-\xi)/\alpha - \exp[-(x-\xi)/\alpha]\}$ $F_X(x) = \exp\{-\exp[-(x-\xi)/\alpha]\}$	$-\infty < x < \infty$	$\mu_X = \xi + 0.5772\,\alpha$ $\sigma 2^2_X = \pi^2\alpha^2/6 = 1.645\alpha^2$; $\gamma_X = 1.1396$
Generalized extreme value	$F_X(x) = \exp\{-[1 - \kappa(x-\xi)/\alpha]^{1/\kappa}\}$ when $0 < \kappa$, $x < (\xi+\alpha/\kappa)$; $\kappa < 0$, $(\xi+\alpha/\kappa) < x$	(σ_X^2 exists for $-0.5 < \kappa$)	$\mu_X = \xi + (\alpha/\kappa)[1 - \Gamma(1+\kappa)]$ $\sigma^2_X = (\alpha/\kappa)^2 \{\Gamma(1+2\kappa) - [\Gamma(1+\kappa)]^2\}$
Weibull	$f_X(x) = (k/\alpha)(x/\alpha)^{k-1} \exp[-(x/\alpha)^k]$ $F_X(x) = 1 - \exp[-(x/\alpha)^k]$	$0 < x$; $0 < k, \alpha$	$\mu_X = \alpha\,\Gamma(1 + 1/k)]$ $\sigma^2_X = \alpha^2 \{\Gamma(1 + 2/k) - [\Gamma(1 + 1/k)]^2\}$
Generalized logistic	$y = [1 - \kappa(x-\xi)/\alpha]^{1/\kappa}$ for $\kappa \neq 0$ $f_X(x) = (1/\alpha)[y^{(1-\kappa)}/(1 + y)]^2$ $F_X(x) = 1/[1 + y]$	$y = \exp[-(x-\xi)/\alpha]$ for $\kappa = 0$ for $\kappa < 0$, $\xi + \alpha/\kappa \le x < \infty$ for $0 < \kappa$, $-\infty < x \le \xi + \alpha/\kappa$	$\mu_X = \xi + \alpha/[1/\kappa - \pi/\sin(\kappa\pi)]$ See Ahmad and others (1998) for σ^2_X.
Generalized Pareto	$f_X(x) = (1/\alpha)[1 - \kappa(x-\xi)/\alpha]^{1/\kappa-1}$ $F_X(x) = 1 - [1 - \kappa(x-\xi)/\alpha]^{1/\kappa}$	for $\kappa < 0$, $\xi \le x < \infty$ for $0 < \kappa$, $\xi \le x \le \xi + \alpha/\kappa$ (γ_X exists for $\kappa > -0.33$)	$\mu_X = \xi + \alpha/(1+\kappa)$ $\sigma^2_X = \alpha^2/[(1+\kappa)^2(1+2\kappa)]$ $\gamma_X = 2(1-\kappa)(1+2\kappa)^{1/2}/(1+3\kappa)$
Halphen			
Type A	$f_X(x) = \frac{1}{2m^\nu K_\nu(2\alpha)} x^{\nu-1} \exp\left[-\alpha\left(\frac{x}{m} + \frac{m}{x}\right)\right]$	for $x > 0$; $m > 0$; $\alpha > 0$; $-\infty < \alpha < \infty$[b]	
Type B	$f_X(x) = \frac{2}{m^{2\nu} ef_\nu(\alpha)} x^{2\nu-1} \exp\left[-\left(\frac{x}{m}\right)^2 + \alpha\left(\frac{x}{m}\right)\right]$	for $x > 0$; $m > 0$; $\nu > 0$; $-\infty < \alpha < \infty$[c]	See Marlat (1956).
Type B^{-1}	$f_X(x) = \frac{2m^{2\nu}}{ef_\nu(\alpha)} x^{-2\nu-1} \exp\left[-\left(\frac{m}{x}\right)^2 + \alpha\left(\frac{m}{x}\right)\right]$	for $x > 0$; $m > 0$; $\nu > 0$; $-\infty < \alpha < \infty$[c]	

[a] Here $Y = \ln(X)$. A three-parameter log-normal distribution with $Y = \ln(X - \xi)$ is also commonly used.

[b] K_ν = modified Bessel function, second kind.

[c] $ef_\nu(\alpha)$ = exponential factorial function.

The Gumbel distribution is a special case of generalized extreme value distribution corresponding to $\kappa = 0$. Here, x is a location parameter, α is a scale parameter, and κ is the important shape parameter. For $\kappa > 0$ the distribution has a finite upper bound at $\xi + \alpha/\kappa$; for $\kappa < 0$, the distribution has a thicker right-hand tail and is unbounded above.

Hosking and others (1985) describe the L–moment procedure that is effective with this distribution. L–moments have been the basis of many regional and index-flood procedures that make use of the generalized extreme value distribution (Hosking and Wallis, 1997). More recently, Martins and Stedinger (2000) present generalized maximum likelihood estimators for the generalized extreme value distribution that are more accurate than L–moment estimators over the range of hydrological interest.

5.3.2.3 Two-parameter Weibull distribution

If W_i are the minimum streamflow in different days of the year, then the annual minimum is the smallest of the W_i, each of which is bounded below by zero. In this case the random variable $X = \min \{W_i\}$ may be described well by the extreme value type III distribution for minima, or the Weibull distribution (see Figure II.5.1 and Table II.5.2). For $k < 1$, the Weibull probability density goes to infinity as x approaches zero, and decays slowly for large values of x. For $k = 1$, the Weibull distribution reduces to the exponential distribution corresponding to $\gamma = 2$. For $k > 1$, the Weibull density function is like a density function of Pearson type III distribution for small values of x and $\alpha_{P3} = k$, but decays to zero faster for large values of x. Parameter estimation methods are discussed in Kite (1988).

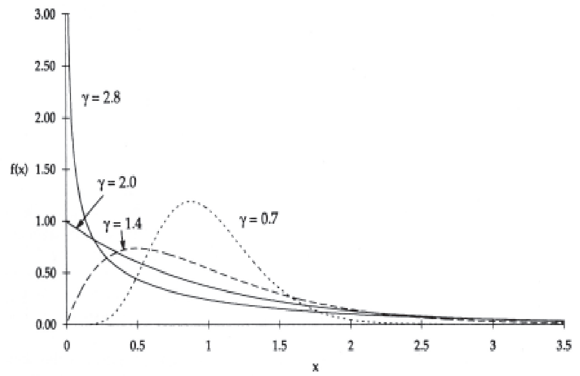


Figure II.5.1. The probability density function for the Pearson type III distribution with lower bound $\zeta = 0$, mean $\mu = 1$ and coefficients of skewness $\gamma = 0.7, 1.4, 2.0$ and 2.8 (corresponding to a gamma distribution and shape parameters $\alpha = 8, 2, 1$ and 0.5, respectively)

5.3.3 Pearson type III family

The Pearson type III (P3) distributions are commonly used to fit a sample of extreme hydrological data. A theoretical description of this distribution can be found in Bobée and Ashkar (1991) and a summary in Maidment's *Handbook of Hydrology*, Chapter 18 (Stedinger and others, 1993). The notations of that publication are used in the following. The probability density function of the P3 distribution, given in Table II.5.2, is defined by three parameters: ζ (location), β (scale) and α (shape). The method of moments considering mean, variance and coefficient of skewness is used by the Interagency Advisory Committee on Water Data (1982) to fit the P3 distribution to data. Caution should be exercised in using moments, as they may yield an upper bound which might be smaller than an observed flood. The method of maximum likelihood can also be used (Pilon and Harvey, 1992). This distribution can be used for both positively and negatively skewed samples.

The log-Pearson type III distribution (LP3) describes a variable x whose logarithm $y = log\ x$ is P3 distributed. This distribution was recommended for the description of floods in the United States of America by the United States Water Resources Council, initially in 1966 and then again by the Interagency Advisory Committee on Water Data in 1982. It was also adopted for use in Canada as one of several other methods (Pilon and Harvey, 1992).

5.3.4 Halphen family: types A, B and B^{-1}

This family of distributions was specifically designed to model floods and more generally, extremes. The probability density function of these distributions (Perreault and others, 1999*a*) are given in Table II.5.2. Perreault and others (1999*b*) presented procedures for estimating parameters, quantiles and confidence intervals for the Halphen distributions. The Gamma and inverse Gamma (x is the inverse Gamma distributed if $y = 1/x$ follows Gamma distributions) are limiting cases of the Halphen distributions.

Although the probability density function of the Halphen distributions are mathematically more complicated than the three-parameter distributions currently used in hydrometeorology, that should not be a serious obstacle for their use in practice, since the Halphen distributions can be applied with the aid of user-friendly software such as HYFRAN (www.ete.inrs.ca/activites/groupes/chaire_hydrol/hyfran.html).

5.3.5 Generalized logistic distribution

The generalized logistic distribution was introduced to the mainstream of the hydrological literature by Hosking and Wallis (1997) and was proposed as the distribution for flood frequency analysis in the United Kingdom (Robson and Reed, 1999). The parameterization is similar to the generalized extreme value distribution, and both have Pareto-like tails for large values of x. The cumulative distribution function of the generalized logistic distribution is given in Table II.5.2, as is the range of the variable. Hosking and Wallis (1997) and Robson and Reed (1999) document how the three parameters of this distribution can be obtained from L–moment estimators.

5.3.6 Generalized Pareto distribution

The generalized Pareto distribution has a very simple mathematical form (see Table II.5.2) and is useful for modelling events that exceed a specified lower bound at which the density function has a maximum ($\kappa < 1$). Examples include daily rainfall depths and all floods above a modest threshold. Hosking and Wallis (1987) discuss alternative estimation procedures. Often the value of the lower bound can be determined by the physical constraints of the situation, so that only two parameters need be estimated. If the physical situation does not dictate the value of the lower bound, then the smallest observation may suffice as an estimator of the lower bound for x.

A very interesting relationship exists between the generalized Pareto distribution and the generalized extreme value distribution. If peaks in a partial duration series arrive as in a Poisson process and have magnitudes described by a generalized Pareto distribution, then the annual maxima greater than the partial duration series threshold have a generalized extreme value distribution with the same value of κ (Stedinger and others, 1993). Wang (1991) and Martins and Stedinger (2001) explore the relative efficiency of the two modelling frameworks.

5.3.7 Non-parametric density estimation method

The non-parametric method does not require either the assumption of the functional form of the overall density function, or the estimation of parameters based on the mean, variance and skew. The non-parametric kernel density estimation requires the selection of a kernel function K, which is a probability density function, and the calculation of a smoothing factor H. Then, using a sample of N observations of the variable x, an approximation of the probability density function for the variable x is obtained by assigning each x_j a probability of $1/N$ and then using the kernel function to spread out that probability around the value of each x_j to obtain the following equation:

$$f(x) = \frac{1}{NH} \sum_{i=1}^{N} K\left(\frac{x - x_i}{H}\right) \quad (5.7)$$

The principle of a kernel estimator as expressed by the above equation is that a kernel of prescribed form, triangular, normal, or Gumbel distribution function is associated with each observation over a specified scale, expressed by H. The weighted sum of these functions constitutes the non-parametric estimate of the density function. The optimal value of H can be determined based on a cross-validation procedure (Adamowski, 1985) and is available in a computer software package (Pilon and others, 1992).

5.4 HYPOTHESIS TESTING

The data series must meet certain statistical criteria such as randomness, independence, homogeneity and stationarity in order for the results of a frequency analysis to be theoretically valid. These statistical criteria are explained in Table II.5.3, where appropriate statistical tests are indicated. A more detailed description of many of these tests can be found in Helsel and Hirsch (1992). Well-known statistical parametric tests such as the *t*-test and the *F*-test are not included in the table because hydrological data series often do not satisfy some conditions for strict applicability of these tests, particularly the assumption of normality, which can adversely impact upon the power of parametric tests (Yue and Pilon, 2004). The tests indicated in the table are of a non-parametric type, which avoids assumptions regarding the underlying parametric distribution of the data. Care should be taken to verify the assumptions underlying the tests, as violation may lead to unreliable results (Yue and others, 2002*a*).

Statistical tests can only indicate the significance of the observed test statistics and do not provide unequivocal findings. It is therefore important to clearly understand the interpretation of the results and to corroborate findings with physical evidence of the causes, such as land use changes. When data do not satisfy the assumptions, then a transformation can often be employed so that the transformed

Table II.5.3. Statistical tests and statistical criteria (after Watt, 1989)

Criterion	*Explanation*	*Applicable statistical tests*
Randomness	In a hydrologic context, randomness means essentially that the fluctuations of the variable arise from natural causes. For instance, flood flows appreciably altered by reservoir operation are unnatural and therefore cannot be considered as random, unless the effect of the regulation is removed first.	No suitable tests for hydrological series are available.
Independence	Independence implies that no observation in the data series has any influence on any following observations. Even if events in a series are random, they may not be independent. Large natural storages, in a river basin, for example, may cause high flows to follow high flows and low flows to follow low flows. The dependence varies with the interval between successive elements of the series: dependence among successive daily flow values tends to be strong, while dependence between annual maximum values is generally weak. Likewise, the elements of annual series of short-duration rainfall may, in practice, be assumed to be independent. In some cases, however, there may be significant dependence even between annual maximum values, for example in the case of rivers flowing through very large storages such as the Great Lakes of North America.	– Anderson as described in Chow (1964). – Spearman rank order serial correlation coefficient as described in NERC (1975).
Homogeneity	Homogeneity means that all the elements of the data series originate from a single population. Elderton (1953) indicated that statistics are seldom obtained from strictly homogeneous material. For instance, a flood series that contains both snowmelt and rainfall floods may not be homogeneous; however, depending on the results of a test, it may be acceptable to treat it as such. When the variability of the hydrological phenomenon is too high, as in the case of extreme precipitation, non-homogeneity tends to be difficult to decipher (Miller, 1972), but non-homogeneity in yearly precipitation sums is easier to detect.	Terry (1952).
Stationarity	Stationarity means that, excluding random fluctuations, the data series is invariant with respect to time. Types of non-stationarity include trends, jumps and cycles. In flood analysis, jumps are generally due to an abrupt change in a basin or river system, such as the construction of a dam. Trends may be caused by gradual changes in climatic conditions or in land use , such as urbanization. Cycles may be associated with long-term climatic oscillations.	– Spearman rank correlation coefficient test for trend (NERC, 1975) – Wald–Wolfowitz (1943) test for trend. No satisfactory method of testing is available for long-period cycles. – Mann–Kendall test for trend (Yue and others, 2002*b*)

observations would meet the criteria required for analysis. Caution is advised in interpolation and extrapolation when data do not meet the assumptions.

5.4.1 Wald–Wolfowitz test for independence and stationarity

Given the data sample of size N $(x_1, \ldots, x_N)$, the Wald–Wolfowitz test considers the statistic R so that:

$$R = \sum_{i=1}^{N-1} x_i x_{i+1} + x_1 x_N \tag{5.8}$$

When the elements of the sample are independent, R asymptotically follows asymptotically normal distribution with mean and variance given by the following equations:

$$\bar{R} = (s^2{}_1 - s_2) / (N - 1) \tag{5.9}$$

$$Var(R) = (s^2{}_2 - s_4) / (N - 1) - \bar{R}^2 \tag{5.10}$$
$$+ (s_1{}^4 - 4s_1{}^2 s_2 + 4s_1 s_3 + s_2{}^2 - 2s_4) / (N - 1)(N - 2)$$

with $s_r = Nm'_r$ and m'_r is the r^th moment of the sample about the origin.

The quantity $(R - \bar{R}) / (Var(R))^{1/2}$ follows a standardized normal distribution (mean 0 and variance 1) and can be used to test at level α the hypothesis of independence by comparing $|n|$ with the standard normal variate $u_{\alpha/2}$ corresponding to a probability of exceedance $\alpha/2$.

5.4.2 **Mann–Kendall test for trend detection**

The Mann–Kendall test is a rank-based non-parametric test for assessing the significance of a trend. The null hypothesis H_0 is that a sample of data ordered chronologically is independent and identically distributed. The statistic S is defined as follows (Yue and others, 2002*b*):

$$S = \sum_{i=1}^{n-1} \sum_{j=i+1}^{n} \mathrm{sgn}(x_j - x_i) \tag{5.11}$$

where

$$\mathrm{sgn}(x) = \begin{cases} 1 & \text{if } x > 0 \\ 0 & \text{if } x = 0 \\ -1 & \text{if } x < 0 \end{cases} \tag{5.12}$$

When $n \geq 40$, the statistic S is asymptotically normally distributed with mean 0 and variance given by the following equation:

$$Var\{S\} = \frac{1}{18}\left[n(n-1)(2n+5) - \sum_t t(t-1)(2t+5)\right] \tag{5.13}$$

where t is the size of a given tied group and $\sum_t$ is the summation over all tied groups in the data sample. The standardized test statistic K is computed by using the following equation:

$$K = \begin{cases} \dfrac{S-1}{\sqrt{Var(S)}} & \text{If } S > 0 \\ 0 & \text{If } S = 0 \\ \dfrac{S+1}{\sqrt{Var(s)}} & \text{If } S < 0 \end{cases} \tag{5.14}$$

The standardized statistic K follows the standard normal distribution with mean zero and variance of one. The probability value P of the statistic K of sample data can be estimated using the normal cumulative distribution function as:

$$P = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{z} e^{-t^2/2} dt \tag{5.15}$$

For independent sample data without trend, the P value should be equal to 0.5. For sample data with large positive trend, the P value should be close to 1.0, whereas a large negative trend should yield a P value close to 0.0. If the sample data are serially correlated, then the data should be pre-whitened and a correction applied to calculate the variance (Yue and others, 2002*b*).

The slope of a trend is estimated as follows:

$$\beta = median\left(\frac{x_i - x_j}{i - j}\right), \forall j < i \tag{5.16}$$

where β is the estimate of the slope of the trend and x_j is the j^{th} observation. An upward trend is represented by a positive value of β and a downward trend is represented by a negative value of β.

5.4.3 **Mann–Whitney test for homogeneity and stationarity (jumps)**

We now consider two samples of size p and q (with $p \leq q$) the combined set of size $N = p + q$ is ranked in increasing order. The Mann–Whitney test considers the following quantities:

$$V = R - p(p+1)/2 \tag{5.17}$$

$$W = pq - V \tag{5.18}$$

where R is the sum of the ranks of the elements of the first sample of size p in the combined series and V and W are calculated from R, p and q. V represents the number of times that an item in sample 1 follows in the ranking an item in sample 2; W can also be computed in a similar way for sample 2 following sample 1.

The test statistic, U, is defined by the smaller of V and W. When $N > 20$, and p, $q > 3$, and under the null hypothesis that the two samples come from the same population, U is approximately normally distributed with mean:

$$\bar{U} = pq/2 \tag{5.19}$$

and variance:

$$Var(U) = \left[\frac{pq}{N(N-1)}\right]\left[\frac{N^3 - N}{12} - \sum T\right] \tag{5.20}$$

with $T = (J^3 - J)/12$, where J is the number of observations tied at a given rank. The summation ΣT is over all groups of tied observations in both samples of size p and q. For a test at a level of significance,

the quantity $|u| = |(U - \bar{U})/Var(U)^{1/2}|$ is compared with the standardized normal quantile $u_{\alpha/2}$ corresponding to a probability of exceedance α /2.

5.4.4 Sample size and length of record

The definition of a stable distribution for estimating future probabilities of occurrence of a hydrological phenomenon requires that the length of record or sample size must be sufficiently long. In estimating daily extreme precipitation, Sevruk and Geiger (1981) report that the length of record needed to obtain a stable distribution is related to the general humidity of the region and its physiographic conditions that determine the variability of the daily precipitation sum. As indicated in Table II.5.3, when the variability of the hydrological phenomenon is too high, difficulties in testing the homogeneity of the hydrological series can arise. When the coefficient of variation of a sample drawn from a skewed distribution is large (large variability), the standard error of the sample coefficient of skewness which is used to fit the assumed distribution will also be large. Sevruk and Geiger (1981) argue that for extreme precipitation frequency analysis a 25-year period of record may be sufficient in humid regions such as the northern Russian Federation, but even a 50-year period is not adequate in other regions where a distinct periodic fluctuation of precipitation exists. According to these authors, a record of 40 to 50 years is, in general, satisfactory for extreme precipitation frequency analysis. Yue and others (2002*a*) and Yue and Pilon (2004) show, as well, how statistical characteristics of the sample and record length can impact upon the power of common statistical tests.

5.4.5 Grubbs and Beck test for detection of outliers

An outlier is defined as a data point that is far from the bulk of the data. The presence of outliers in a data sample can cause difficulties when attempting to fit a distribution to the sample. There may exist high or low outliers, or both, in a sample, and these can have different impacts on the frequency analysis. Although the problem of treating outliers is still subject to much discussion, certain procedures have been used in hydrology for their identification and treatment, such as those described by the United States Water Resources Council (1981) for flood frequency analysis or by Sevruk and Geiger (1981) for extreme precipitation.

The Grubbs and Beck test for the detection of outliers is the test that is recommended by the United States Water Resources Council (1981). To apply this test, the assumption must be made that the logarithms or some other function of the hydrological series are normally distributed because the test is applicable only to samples from a normal population. It is common to make the simple assumption used by the United States Water Resources Council that the logarithms of the sample values are normally distributed. To apply the Grubbs and Beck test, the following two quantiles are calculated:

$$X_H = exp\ (\bar{x} + K_N s) \tag{5.21}$$

$$X_L = exp\ (\bar{x} + K_N s) \tag{5.22}$$

where $\bar{x}$ and s are the mean and standard deviation of the natural logarithms of the sample, respectively, and K_N is the Grubbs and Beck statistic tabulated for various sample sizes and significance levels. At the 10 per cent significance level, the following polynomial approximation proposed by Pilon and Harvey (1992) can be used for estimating the tabulated values:

$$K(N) = -3.62201 + 6.2844N^{1/4} - 2.49835N^{1/2} + 0.491436N^{3/4} - 0.037911N \tag{5.23}$$

where N is the sample size. In applying the Grubbs and Beck test, any sample values greater than X_H are considered to be high outliers and those less than X_L are considered to be low outliers. For $5 \leq N \leq 150$, $K(N)$ can be computed from the following equation (Stedinger and others, 1993):

$$K(N) = -0.9043 + 3.345\sqrt{log\ (N)} - 0.4046\ log\ (N) \tag{5.24}$$

5.4.6 Bayesian procedures

While the frequency estimation of probability is based on the idea of an experiment that can be repeated several times, the Bayesian approach is based on a personal assessment of probability and provides an opportunity to take into account any information that is available, by means of the prior distribution. Unlike classical models, Bayesian models consider the parameters of the problem as random variables rather than fixed values. For example, in the case of the detection of shifts in the mean of a time series, classical statistical methods assume knowledge of the time of the possible shift. The Bayesian approach, however, does not make any assumptions concerning knowledge of the time of the shift. This allows the approach to make inferences on its characteristics, such as the change point and the amount of shift.

Perreault and others (1999*c*) and other authors have presented Bayesian models for the detection of a single shift in the mean. Perreault and others (2000) presented a method for a change in variability and applied it to hydrological data, while Asselin and others (1999) presented a bivariate Bayesian model for the detection of a systematic change in the mean. A complete description of the Bayesian statistical inference theory is presented in Box and Tiao (1973).

5.5 POPULATION STATISTICS AND PARAMETER ESTIMATION

Assuming that extreme events are described properly by some family of distributions, a hydrologist's task is to estimate the parameters of the distribution so that required quantiles and expectations can be calculated with the fitted model. The statistical and hydrological literature contains many methods and philosophies for estimating the parameters of different distributions: those most commonly employed are outlined below.

5.5.1 Parameter calculation methods

Perhaps the simplest approach is the method of moments, which computes estimates of the parameters so that the theoretical moments of a distribution match the computed sample moments. The recommended procedure for federal agencies in the United States (Thomas, 1985; Interagency Advisory Committee on Water Data, 1982) uses the moments of the logarithms of the floods flows $X = log\ Q$.

A variation on the method of moments, which has proved effective in hydrology with the generalized extreme value distribution, is the method of probability-weighted moments or equivalently L–moments (Hosking and others, 1985; Hosking and Wallis, 1997). Probability-weighted moments or the corresponding L–moments provide a different way to summarize the statistical properties of hydrological datasets (Hosking, 1990). An advantage of L–moment estimators are that they are linear combinations of the observations and thus do not involve squaring and cubing the observations. As a result, the L-coefficient of variation and L-skewness are almost unbiased, whereas the product-moment estimators of the coefficient of variation and coefficient of skewness are highly biased and highly variable (Vogel and Fennessey, 1993). This is of particular value for regionalization procedures, which is further discussed in 5.9.

L–moments are another way to summarize the statistical properties of hydrological data based on linear combinations of the original data (Hosking, 1990). Recently, hydrologists have found that regionalization methods that use L–moments are superior to methods that use traditional moments. They have also worked well for fitting some distributions with on-site data (Hosking and others, 1985). The first L–moment is the arithmetic mean:

$$\lambda_1 = E[X] \tag{5.25}$$

Let $X_{(i|n)}$ be the i^{th} largest observation in a sample of size n (i = 1 corresponds to the largest). Then, for any distribution, the second L–moment is a description of scale based on the expected difference between two randomly selected observations:

$$\lambda_2 = (1/2)\ E[X_{(1|2)} - X_{(2|2)}] \tag{5.26}$$

Similarly, L–moment measures of skewness and kurtosis use:

$$\lambda_3 = (1/3)\ E[X_{(1|3)} - 2\ X_{(2|3)} + X_{(3|3)}] \tag{5.27}$$

$$\lambda_4 = (1/4)\ E[X_{(1|4)} - 3\ X_{(2|4)} + 3\ X_{(3|4)} - X_{(4|4)}] \tag{5.28}$$

Just as product moments can be used to define dimensionless coefficients of variation and skewness, L–moments can be used to define a dimensionless L-coefficient of variation and an L–coefficient of skewness (Table II.5.4). L–moment estimators have often been computed based on an intermediate statistics called probability-weighted moments (Hosking, 1990; Hosking and Wallis, 1997; Stedinger and others, 1993). Many early studies used probability-weighted moment estimators based on plotting positions (Hosking and others,

Table II.5.4. Dimensionless statistics used to describe distributions (product–moment and L–moment ratios)

Name	*Denotation*	*Definition*
Product–moment ratios		
Coefficient of variation	CV_X	σ_X/μ_X
Coefficient of skewness[a]	γ_X	$E[(X - \mu_X)^3] / \sigma_X{}^3$
Coefficient of kurtosis[b]	–	$E[(X - \mu_X)^4] / \sigma_X{}^4$
L–moment ratios[c]		
L–coefficient of variation	L–CV, τ_2	λ_2/λ_1
L–coefficient of skewness	L–skewness, τ_3	λ_3/λ_2
L–coefficient of kurtosis	L–kurtosis, τ_4	λ_4/λ_2

[a] Some texts define $\beta_1 = [\gamma_x]^2$ as a measure of skewness.
[b] Some texts define the kurtosis as $\{E[(X - \mu_x)^4]/\sigma_x^4 - 3\}$; others use the term excess kurtosis for this difference because the normal distribution has a kurtosis of 3.
[c] Hosking (1990) uses τ instead of τ_2 to represent the L–CV ratio.

1985); these were later found to lack the consistency and invariance required of such estimators (Hosking and Wallis, 1995; Fill and Stedinger, 1995), so that subsequent work has shifted to use the unbiased probability-weighted moment estimators. Direct estimation of unbiased L–moments from a sample is described by Wang (1996).

A method that has very strong statistical motivation is maximum likelihood. It chooses the parameters which make a fitted distribution as consistent as possible, in a statistical sense, with the observed sample. Maximum likelihood estimators are discussed in general statistics textbooks and are recommended for use with historical and paleoflood records because of their ability to make particularly efficient use of censored and categorical datasets.

Non-parametric methods can be employed to estimate the flood–flow frequency relationship, offering the advantage that one need not assume that floods are drawn from a particular parametric family of distributions. These methods have been adopted in Canada (Pilon and Harvey, 1992).

5.5.2 Use of logarithmic transformations

When data vary widely in magnitude, which frequently occurs in water quality monitoring, the sample product-moments of the logarithms of data are often employed to summarize the characteristics of a dataset or to estimate distribution parameters. A logarithmic transformation is an effective vehicle for normalizing values that vary by order of magnitude, and for preventing occasionally large values from dominating the calculation of product-moment estimators. However, the danger of using logarithmic transformations is that unusually small observations or low outliers are given greatly increased weight. This is of concern when large events are of interest and when small values are poorly measured. Small values may reflect rounding errors or may be reported as zero when they fall below a certain threshold.

5.5.3 Historical information

In addition to a relatively brief period of systematic measurements, there may be additional historical information available that pertains, for example, to the magnitude of floods prior to the commencement of the systematic collection of records. A gauging station might have only 20 years of measurement records as of 1992, yet it might be known that in 1900 a flood occurred with a peak which exceeded any flood measured and was also the greatest flood since the community was established in 1860. The magnitude of this flood and the knowledge that the other floods from 1860 to 1992 were less than the flood of 1900 should and can be used in the frequency analysis. In other instances, it may only be known that a certain number of floods from 1860 to 1972 exceeded a certain threshold. This is also historical information and should be included in the frequency analysis. Different processes generate historical and physical paleoflood records. Floods leaving a high-water mark are the largest to have occurred during the corresponding period, whereas slackwater sediment deposits in protected areas can provide evidence of the magnitude of a number of large floods.

Apart from the routine monitoring of streamflow, certain floods may be recorded simply because they exceed a perception level and have sufficiently disrupted human activities for their occurrence to have been noted, or for the resultant physical or botanical damage to be available with which to document the event (Stedinger and Baker, 1987; Wohl, 2000). Several methods can be used to incorporate historical information into the estimation of the parameters of the mathematical distribution function. They are historically adjusted weighted moments, maximum likelihood, the expected moments algorithm and the non-parametric method (Cohn and others, 2001; England and others, 2003; Griffis and others, 2004). It has been shown that the maximum likelihood method makes more efficient use of the additional information than historically weighted moments. Maximum likelihood estimators and expected moments algorithms are both very flexible and appear to be equally efficient with the LP3 distribution for which expected moments algorithms were developed, though maximum likelihood estimators often have convergence problems with those distributions.

5.5.4 Record augmentation

It is often possible to effectively extend a short record using a longer record from a nearby station with which observations in the the short record are highly correlated. In particular, a long series from a nearby station can be used to improve estimates of the mean and variance of the events that occur at the short-record site. For this purpose, it is not necessary to actually construct the extended series; one only needs the improved estimates of the moments. This idea of record augmentation is developed in Matalas and Jacobs (1964); see also the Interagency Advisory Committee on Water Data

(1982), (Appendix 7). Recent improvements and a discussion of the information gain are provided by Vogel and Stedinger (1985). In other instances, a longer series can be created that will be employed in simulation or will be archived. The idea of using record extension to ensure that generated flows have the desired mean, variance and correlations is developed by Hirsch (1982), Vogel and Stedinger (1985) and, where multivariates are concerned, by Grygier and others (1989).

5.5.5 Analysis of mixed populations

A common problem in hydrology is that annual maximum series are composed of events that may arise from distinctly different processes. For example, precipitation may correspond to different storm types in different seasons, such as summer thunderstorms, winter frontal storms, and remnants of tropical hurricanes or snowmelt. Floods arising from these different types of events may have distinctly different distributions. Waylen and Woo (1982) examined summer runoff and winter snowmelt floods separately. Vogel and Stedinger (1984) studied summer rainfall and winter ice-jam-affected floods. Hirschboeck and others (2000) considered the categorization of different floods above a specific threshold into classes based on the prevailing synoptic weather pattern; this resulted in a mixed-population flood analysis using a partial duration series framework. In some mountainous regions in small basins, summer thunderstorms produce the largest floods of record, but snowmelt events produce most of the maximum annual events. In such instances, as illustrated by Waylen and Woo (1982), separation of the flood record into separate series can result in a better estimate of the probability of extreme events because the data describing phenomena that produce those large events is better represented in the analysis.

Suppose that the annual maximum series M_t is the maximum of the maximum summer event S_t and the maximum winter event W_t:

$$M_t = max\ \{S_t,\ W_t\} \qquad (5.29)$$

Here S_t and W_t may be defined by a rigidly specified calendar period, a loosely defined climatic period, or the physical and meteorological characteristics between the phenomena that generated the observations.

If the magnitudes of the summer and winter events are statistically independent, meaning that knowing one has no effect on the conditional probability distribution of the other, the probability distribution for the annual maximum event M is given by (Stedinger and others, 1993):

$$F_M\ (m) = P[M = max(S,\ W) \leq m] = F_S(m)\ F_W\ (m) \qquad (5.30)$$

For two or more independent series of events contributing to an annual maximum, the distribution of the maximum is easily obtained. If several statistical-dependent processes contribute to an annual maximum, the distribution of the maximum is much more difficult to calculate from the distributions of the individual series. An important issue is that of deciding whether it is advisable to model several different component flood series separately, or whether it is just as reasonable to model the composite maximum annual series directly. If several series are modelled, then more parameters must be estimated, but more data are available if the annual maximum series or the partial duration series for each type of event is available.

The idea of the mixing of two distributions led to the devolopment of a two-component extreme value ß distribution by Rossi and others (1984), which corresponds to the maximum of two independent EV1 distributions. It can be thought of as the maximum of two flood processes in a partial duration series, each with Poisson arrivals and exponentially distributed flood peaks. Generally, one of the two distributions is thought of as describing the bulk of the data, and the other as the outlier distribution. Because the model has four parameters, it is very flexible (Beran and others, 1986). Therefore, if only the annual maximum series are used, regional estimation methods are essential to resolve the values of all four parameters, making regional two-component extreme value estimators an attractive option. The two-component extreme value distribution has been successfully employed as the basis of index flood procedures (Frances, 1998; Gabriele and Villani, 2002). The non-parametric distribution (Adamowski, 1985) and Wakeby distribution (Pilon and Harvey, 1992) can also be used to model the mixture distribution.

5.5.6 Frequency analysis and zeros

Low-flow series often contain years with zero values, while some sites' maximum series may also contain zero values for some sites. In some arid areas, zero flows are recorded more often than non-zero flows. Streamflows recorded as zero imply either that the stream was completely dry, or that the actual streamflow was below a recording or detection limit. This implies that some low-flow series are censored datasets. Zero values should not simply be

ignored and do not necessarily reflect accurate measurements of the minimum flow in a channel. Based on the hydraulic configuration of a gauge and on knowledge of the rating curve and recording policies, it is possible to determine the lowest discharge that can be reliably estimated and would not be recorded as a zero. The plotting position method and the conditional probability model are reasonable procedures for fitting a probability distribution with datasets containing recorded zeros. The graphical plotting position approach, without a formal statistical model, is often sufficient for low-flow frequency analyses. The low-flow frequency curve can be defined visually and the parameters of a parametric distribution can be estimated by using probability-plot regression as described by Kroll and Stedinger (1996) and Stedinger and others (1993), or by using non-parametric methods.

5.6 PROBABILITY PLOTS AND GOODNESS-OF-FIT TESTS

5.6.1 Plotting positions and probability plot

Initial evaluation of the adequacy of a fitted probability distribution is best done by generating a probability plot of the observations. When the sorted observations are plotted against an appropriate probability scale, except for sampling fluctuation, they fall approximately on a straight line.

Such a plot serves both as an informative visual display of the data and a check to determine whether the fitted distribution is consistent with the data.

Such plots can be generated with special commercially available probability papers for some distributions, including the normal, two-parameter log-normal and Gumbel distributions, all of which have a fixed shape. Thanks to modern software, however, it is generally easier to generate such plots without the use of special papers (Stedinger and others, 1993). The i^{th} largest observed flood $x_{(i)}$ is plotted versus the estimated flood flow associated with the exceedance probability, or probability-plotting position q_i, assigned to each ranked flood $x_{(i)}$; $x_{(1)} > x_{(2)} > \ldots > x_{(n)}$. The exceedance probability of the i^{th} largest flood $x_{(i)}$ can be estimated by any of several reasonable formulae. Three commonly used are the Weibull formula with $p_i = i / (n + 1)$, the Cunnane formula with $p_i = (i – 0.40) / (n + 0.2)$, and the Hazen formula with $p_i = (i – 0.5) / n$. Cunnane (1978) and Adamowski (1981) provide a discussion of the plotting position issue. Plotting positions for records that contain historical information is developed in Hirsch and Stedinger (1987). Hydrologists should remember that the actual exceedance probability associated with the largest observation in a random sample has a mean of $1/(n+1)$ and a standard deviation of nearly $1/(n+1)$ (Stedinger and others, 1993); thus all of the plotting positions give only crude estimates of the relative range of exceedance probabilities that could be associated with the largest events (Hirsch and Stedinger, 1987).

5.6.2 Goodness-of-fit tests

Several rigorous statistical tests are available and are useful in hydrology to determine whether it is reasonable to conclude that a given set of observations was drawn from a particular family of distributions (Stedinger and others, 1993). The Kolmogorov–Smirnov test provides bounds within which every observation on a probability plot should lie if the sample is actually drawn from the assumed distribution (Kottegoda and Rosso, 1997). The probability plot correlation test is a more effective test of whether a sample has been drawn from a postulated distribution (Vogel and Kroll, 1989; Chowdhury and others, 1991). Recently developed L–moments can be used to assess if a proposed Gumbel-, generalized extreme value- or normal distribution is consistent with a dataset (Hosking, 1990; Chowdhury and others, 1991). Discussion of the development and interpretation of probability plots is provided by Stedinger and others, (1993) and Kottegoda and Rosso (1997).

5.6.3 Information criteria

Many approaches have been suggested for the comparison of flood distributions. Goodness-of-fit tests have been applied to assess the suitability of different probability distributions for describing annual maximum flow series, and to evaluate simulated samples in the case of simulation studies. These tests establish which distributions are, in general, the most appropriate for flood modeling. To assess the quality of a fitted model, Akaike (1974) introduced an information criterion called AIC, which stands for Akaike information criterion. It can be adapted to many different situations and consists in minimizing an information measure. The information criterion is defined as follows:

$$AIC(f) = -2\log L(\hat{\theta}, x) + 2k \tag{5.31}$$

where $L(\hat{\theta}, x)$ is the likelihood function, and k is the number of parameters. According to Akaike (1974),

the model that better explains the data with the least number of parameters is the one with the lowest Akaike information criterion. To select an appropriate model, some compromises between the goodness of fit and the complexity of the model must be accepted. Alone, the Akaike information criterion is not appropriate for model selection.

A Bayesian extension of the minimum Akaike information criterion concept is the Bayesian information criterion called BIC. It is defined as follows:

$$BIC(f) = -2log\, L(\hat{\theta}, x) + k\, log(n) \qquad (5.32)$$

where $L(\hat{\theta}, x)$ the likelihood function, k is is the number of parameters and n is the sample size. The Bayesian information criterion is also a parsimony criterion. Of all the models, the one with the lowest Bayesian information criterion is considered to be best. The Schwarz method (1978) is often used to obtain the Bayesian information criterion . However, this method can also be used to get an asymptotic approximation of a Bayes factor. Furthermore, it can be combined to an a priori probability distribution to obtain the a posteriori probability for each distribution of a given set of distributions. Bayesian information criteria have not yet been used much in hydrology, however. The above-mentioned methods, which merit broader use, are available in HYFRAN software. Ozga-Zielinska and others (1999) developed a computer package for calculating design floods when a sufficiently long period of record is available. There are many other computer packages, including those listed in HOMS.

5.7 RAINFALL FREQUENCY ANALYSIS [HOMS I26, K10, K15]

The frequency of occurrence of rainfall of different magnitudes is important for various hydrological applications. In particular, rainfall frequency analyses are used extensively to plan and design engineering works that control storm runoff, such as dams, culverts, urban and agriculture drainage systems. This is because, in most cases, good-quality flow data of a length adequate for the reliable estimation of floods are generally limited or unavailable at the location of interest, while extensive precipitation records are often available. In general, there are two broad categories of approaches for estimating floods from precipitation data: those employing the statistical analysis of precipitation data and those based on the deterministic estimation of the so-called probable maximum precipitation (PMP). While it has been used worldwide for the design of various large hydraulic structures, probable maximum precipitation does not provide probability estimates for risk assessment work. The main part of this section focuses, therefore, on statistical rainfall estimation methods that can provide both flood magnitudes and associated probabilities; the second part deals with the estimation of extreme rainfall. The theory and applications of PMP have been well documented in hydrological and engineering literature such as the *Manual for Estimation of Probable Maximum Precipitation* (WMO-No. 332) and NRCC (1989); and are summarized in 5.7.5.6.

The main objective of rainfall frequency analysis is to estimate the amount of precipitation falling at a given point or over a given area for a specified duration and return period. Results of this analysis are often summarized by intensity–duration–frequency relationships for a given site or are presented in the form of a precipitation frequency atlas, which provides rainfall accumulation depths for various durations and return periods over the region of interest. For instance, estimates of rainfall frequencies for various durations, ranging from 5 minutes to 10 days, and return periods from 1 to 100 years are available. Such data can be found for the United States in the US Weather Service and Atlas series of the National Oceanic and Atmospheric Administration (Frederick and others, 1977), for Australia in *Australian Rainfall and Runoff: A Guide to Flood Estimation* (Pilgrim, 1998), for Canada in the *Rainfall Frequency Atlas for Canada* (Hogg and Carr, 1985) or in the *Handbook on the Principles of Hydrology* (Gray, 1973) and for the United Kingdom in the *Flood Estimation Handbook* (Institute of Hydrology, 1999).

Basic considerations of frequency analysis of hydrological data are discussed in 5.1 to 5.6, whereas special applications for rainfall analysis are covered in 5.7. The statistical methods described herein apply to storm or other short-duration rainfall data. Similar methods are used for flood peaks, flood volumes, low flows, droughts and other extreme events. In particular, the selection of distribution types for extremes of precipitation is discussed by WMO (1981).

5.7.1 Assessment of rainfall data for frequency analysis

Rainfall data used for frequency analysis are typically available in the form of annual maximum series, or are converted to this form using continuous records of hourly or daily rainfall data. These

series contain the largest rainfall in each complete year of record. An alternative data format for precipitation frequency studies is partial duration series, also referred to as peaks over threshold data, which consist of all large precipitation amounts above certain thresholds selected for different durations. The difference in design rainfall estimates using annual maximum and partial duration series was found to be important for short return periods of two to five years but insignificant for long return periods of ten years or longer (Chow, 1964; Stedinger and others, 1993).

As for any statistical analyses, both the quantity and quality of the data used are important. The precipitation data should be collected for a long period of time. A sufficiently long record of precipitation data provides a reliable basis for frequency determinations. It is known that a data sample of size n, in the absence of a priori distributional assumptions, can furnish information only about exceedance probabilities greater than approximately 1/n (NRC, 1988). It is a common rule of thumb to restrict extrapolation of at-site quantile estimates to return periods (years) of up to twice as long as the record length (NRCC, 1989). Hence, long-term precipitation data are extremely valuable for determining statistically based rainfall estimates of reasonable reliability, especially for extreme rainfalls with high return periods, such as those greater than 100 years.

The quality of precipitation data may affect its usability and proper interpretation in flood frequency analysis studies. Precipitation measurements are subject to both random and systematic errors (Sevruk, 1985). The random error is due to irregularities of topography and microclimatical variations around the gauge site. Random errors are also caused by inadequate network density to account for the natural spatial variability of rainfall. The systematic error in point precipitation measurement is, however, believed to be the most important source of error. The largest systematic error component is considered to be the loss due to wind field deformation above the orifice of elevated precipitation gauges. Other sources of systematic error are wetting and evaporation losses of water that adheres to the funnel and measurement container, and rain splash. A broader discussion of systematic errors and their correction is contained in Volume I, 3.3.6, of this Guide.

As rainfall data are collected at fixed observation times, for example clock hours, they may not provide the true maximum amounts for the selected durations. For example, studies of thousands of station-years of rainfall data indicate that multiplying annual maximum hourly or daily rainfall amounts for a single fixed observational interval of 1 to 24 hours by 1.13 will yield values close to those to be obtained from an analysis of true maxima. Lesser adjustments are required when maximum observed amounts are determined from 2 or more fixed observational intervals as indicated in Table II.5.5 (NRCC, 1989). Thus, maximum 6- and 24-hour amounts determined from 6 and 24 consecutive fixed one-hour increments require adjustment by factors of only 1.02 and 1.01, respectively. These adjustment factors should be applied to the results of a frequency analysis of annual maximum series to account for the problem of fixed observational times (NRCC, 1989).

Table II.5.5. Adjustment factor for daily observation frequency

Number of observations/ days	1	2	3–4	5–8	9–24	> 24
Adjustment factor	1.13	1.04	1.03	1.02	1.01	1.00

For frequency analysis studies, it is necessary to check precipitation data for outliers and consistency. As noted in 5.4.5, an outlier is an observation that departs significantly from the general trend of the remaining data. Procedures for treating outliers require hydrological and mathematical judgment (Stedinger and others, 1993). In the context of regional analysis of precipitation, the outliers could provide critical information for describing the upper tail of the rainfall distribution. Hence, high outliers are considered to be historical data if sufficient information is available to indicate that these outlying observations are not due to measurement errors. Regarding data inconsistency, there are many causes. Changes in gauging instruments or station environment may cause heterogeneity in precipitation time series. Data from the gauge sites located in forest areas may not be compatible with those measured in open areas. Measurements in the valley and mountain stations and at various altitudes will not provide identical information regarding precipitation characteristics. Therefore, care must be used in applying and combining the precipitation data.

5.7.2 **At-site frequency analysis of rainfall**

A frequency analysis can be performed for a site for which sufficient rainfall data are available. Similar to flood frequency analysis, rainfall frequency

analysis is also based on annual maximum series or partial duration series (for example, Wang, 1991; Wilks, 1993). Arguments in favor of either of these techniques are contained in the literature (NRCC, 1989; Stedinger and others, 1993). Owing to its simpler structure, the annual maximum series-based method is more popular. The partial duration analysis, however, appears to be preferable for short records, or where return periods shorter than two years are of interest. The choice of an appropriate technique should depend on the purpose of the analysis and characteristics of the available data in terms of both quantity and quality. Improved reliability of the results can be generally achieved with the use of sophisticated and comprehensive analysis methods. Virtually all hydrological estimates are subject to uncertainty. Therefore, it is often advisable to produce estimates using two or more independent methods and to perform a sensitivity analysis to gain information regarding the potential reliability of results.

Briefly, the steps below should be followed to determine the frequency distribution of annual maximum rainfall for a given site:

(a) Obtain a data sample and perform an assessment of data quality based on hydrological and statistical procedures;
(b) Select a candidate distribution model for the data and estimate the model parameters;
(c) Evaluate the adequacy of the assumed model in terms of its ability to represent the parent distribution from which the data were drawn.

The assessment of data quality is an important step in all statistical computations. The basic assumption in precipitation frequency analysis is that the data are independent and identically distributed. As mentioned above, precipitation measurements could be subject to various sources of error, inconsistency and heterogeneity. Detailed examination and verification of the raw data are needed to identify invalid data in the record caused by instrument malfunction and/or human error. Standard statistical tests are available to verify serial independence, stationarity and homogeneity of the data series (see 5.4).

There is no general agreement as to which distribution or distributions should be used for precipitation frequency analysis. A practical method for selecting an appropriate distribution is by examining the data with the use of probability plots. Probability plots, which require the use of a plotting position formula, are an effective tool to display graphically the empirical frequency distribution of the data and to assess whether the fitted distribution appears consistent with the data. There are several plotting-position formulae available in practice (see 5.6 and Nguyen and others, 1989) among which the Hazen, Weibull, and Cunnane formulas are the most popular. The differences between these three formulae are small for observations that are neither the largest nor the smallest; however, they can be significant for the largest three or four values in the data series (Stedinger and others, 1993). An alternative method for making a good choice among different distributions is based on the L–moment diagram (Stedinger and others, 1993).

Common distributions that have been applied to the analysis of annual maximum series include the Gumbel, generalized extreme value, log-normal, and log-Pearson type III distributions. Among these distributions, the generalized extreme value and its special form, the Gumbel distribution, have received dominant applications in modelling the annual maximum rainfall series. The Gumbel distribution was found, however, to underestimate the extreme precipitation amounts (Wilks, 1993). Adamowski and others, (1996) have shown that Canadian precipitation intensity data for various durations do not appear to be drawn from a Gumbel distribution. Studies using rainfall data from tropical and non-tropical climatic regions (Nguyen and others, 2002; Zalina and others, 2002) also suggest that a three-parameter distribution can provide sufficient flexibility to represent extreme precipitation data. In particular, the generalized extreme value distribution has been found to be the most convenient, since it requires a simpler method of parameter estimation and is more suitable for regional estimation of extreme rainfalls at sites with limited data or with no data (Nguyen and others, 2002). When the return periods associated with frequency-based rainfall estimates greatly exceed the length of record available, discrepancies between commonly used distributions tend to increase.

Many methods for estimating distribution parameters are available in the hydrological and statistical literature. The simplest method is the method of moments that provides parameter estimates indicating that the theoretical moments are equal to the computed sample moments. An alternative method for estimating parameters is based on the sample L–moments. These are found to be less biased than traditional moment estimators, and are thus better suited for use with small sample sizes. The L–moment method has proved effective in the estimation of the generalized extreme value distribution parameters (Stedinger and others, 1993). Another method is the method of maximum likelihood. This method provides estimators with very

good statistical properties in large samples, but the estimators are often not available in closed form and thus must be computed using an iterative numerical method.

The reliability of precipitation frequency estimates depends on how well the fitted model represents the parent distribution. Several goodness-of-fit criteria can be used to test whether a selected distribution is consistent with a particular data sample (NRCC, 1989; Stedinger and others, 1993; ASCE, 1996). As mentioned above, probability plots are extremely useful in the assessment of the adequacy of fitted distributions. The assessment is performed by plotting observed rainfall data versus plotting-position estimates of exceedance probability on a specialized plotting paper. The estimated distribution is plotted on the same graph. Goodness of fit is judged by inspection. More rigorous statistical tests such as the Kolmogorov–Smirnov, probability plot correlation and L–moment tests are available, allowing quantitative judgment of goodness of fit. However, the selection of the distribution that best fits each dataset is not a recommended approach for frequency analysis (Stedinger and others, 1993; ASCE, 1996). The use of the best-fitting distribution for each data sample provides frequency estimates that are too sensitive to the sampling variations in the data and the period of record available. Current distribution selection procedures adopted by many countries are based on a combination of regionalization of some parameters and split-sample Monte-Carlo evaluations of different estimation methods to find distribution-estimation procedure combinations that give reliable quantile and risk estimates (Stedinger and others, 1993; ASCE, 1996).

5.7.3 **Regional rainfall frequency analysis**

Even a long record may be a relatively small sample of a climatic regime. A better measure of the regime at a station may be given by a smoothed map, which includes information from nearby stations that can influence point data, and thus broadens the sample. The degree of smoothing should be consistent with the spacing of observation stations and the sampling error of the stations. Too little smoothing tends to confound sampling error with spurious regional variation.

Rainfall frequency atlases have been produced by interpolation and smoothing of at-site frequency analysis results. Regional frequency analysis, which involves data from many sites, has been shown to reduce the uncertainties in quantile estimation of extreme events (Hosking and Wallis, 1988). Similarly to regional flood analyses, the following issues should be addressed when conducting regional precipitation analyses: the selection and verification of homogeneous regions, and regional distribution parameters. Several regional estimation methods have been suggested, among which identification of the regional probability distribution and the estimation of the index-flood procedure for use with the annual maximum series are the most popular. For example, Schaefer (1990) used the index flood methodology to conduct regional analyses of annual maximum precipitation data in Washington State. It has been shown that climatically homogeneous regions can be defined based on the mean annual precipitation. Further, it was found that the coefficients of variation and skew of annual maximum rainfalls vary systematically with the mean annual precipitation. Hence, all sites within a homogeneous region could be characterized by a specific three-parameter probability distribution, such as the generalized extreme value, having fixed values of the coefficients of variation and skew. However, the use of mean annual precipitation as an index variable may not be appropriate for other regions with different climatic or topographic conditions. For instance, the median of annual maximum rainfalls at a site was recommended as the index variable for regional estimation of extreme rainfalls in the United Kingdom of Northern Ireland and Great Britain (Institute of Hydrology, 1999). In general, one of the main difficulties in the application of this technique is related to the definition of homogeneous regions. Various methods have been proposed for determining regional homogeneity, but there is no generally accepted procedure in practice (Fernandez Mill, 1995; Nguyen and others, 2002).

Another regional rainfall frequency analysis method is the station-year method. This method attempts to enlarge the sample size by pooling records from a number of stations into a single large sample of size equal to the number of station years of record. Hence, when applying the station-year method, it is not advisable to estimate rainfall amounts for a site for return periods that are much longer than the length of record at any of the stations. However, the method may yield more reliable estimates if the stations can be considered to be meteorologically homogeneous, rather than using only the data originating from one site. Further, the effect of interstation correlation should be investigated because spatial correlation between samples tends to significantly reduce the number of station years.

Table II.5.6. World's greatest observed point rainfalls

Duration	*Depth (mm)*	*Location*	*Date*
1 min	38	Barot, Guadeloupe	26 November 1970
8 min	126	Fussen, Bavaria	25 May 1920
15 min	198	Plumb Point, Jamaica	12 May 1916
20 min	206	Curtea-de-Arges, Romania	7 July 1889
42 min	305	Holt, Misssouri, United States	22 June 1947
1 h 00 min	401	Shangdi, Nei Monggol, China	3 July 1975
2 h 10 min	483	Rockport, West Virginia, United States	18 July 1889
2 h 45 min	559	D'Hanis, Texas, United States	31 May 1935
4 h 30 min	782	Smethport, Pennsylvania, United States	18 July 1942
6 h	840	Muduocaidang, Nei Monggol, China	1 August 1977
9 h	1087	Belouve, Reunion Island	28 February 1964
10 h	1400	Muduocaidang, Nei Monggol, China	1 August 1977
18 h 30 min	1689	Belouve, Reunion Island	28–29 February 1964
24 h	1825	Foc Foc, Reunion Island	7–8 January 1966
2 days	2467	Aurere, Reunion Island	7–9 April 1958
3 days	3130	Aurere, Reunion Island	6–9 April 1958
4 days	3721	Cherrapunji, India	12–15 September 1974
5 days	4301	Commerson, Reunion Island	23–27 January 1980
6 days	4653	Commerson, Reunion Island	22–27 January 1980
7 days	5003	Commerson, Reunion Island	21–27 January 1980
8 days	5286	Commerson, Reunion Island	20–27 January 1980
9 days	5692	Commerson, Reunion Island	19–27 January 1980
10 days	6028	Commerson, Reunion Island	18–27 January 1980
11 days	6299	Commerson, Reunion Island	17–27 January 1980
12 days	6401	Commerson, Reunion Island	16–27 January 1980
13 days	6422	Commerson, Reunion Island	15–27 January 1980
14 days	6432	Commerson, Reunion Island	15–28 January 1980
15 days	6433	Commerson, Reunion Island	14–28 January 1980
31 days	9300	Cherrapunji, India	1–31 July 1861
2 months	12767	Cherrapunji, India	June–July 1861
3 months	16369	Cherrapunji, India	May–July 1861
4 months	18738	Cherrapunji, India	April–July 1861
5 months	20412	Cherrapunji, India	April–August 1861
6 months	22454	Cherrapunji, India	April–September 1861
11 months	22990	Cherrapunji, India	January–November 1861
1 year	26461	Cherrapunji, India	August 1860–July 1861
2 years	40768	Cherrapunji, India	1860–1861

Revised: 29 November 1991, US National Weather Service, US Department of the Interior Bureau of Reclamation, Australian Bureau of Meteorology

Owing to the latter and the spatial heterogeneity of climatic data, this approach is seldom used in practice.

5.7.4 Frequency analysis of area-averaged rainfall

In general, a catchment-average design rainfall is often required for design flood estimation, especially for large drainage basins. For instance, when the area of a basin exceeds about 25 km^2, rainfall observations at a single station, even if at the centre of the catchment, will usually be inadequate for the design of drainage works. All rainfall records within the catchment and its immediate surroundings must be analysed to take proper account of the spatial and temporal variation of rainfall over the basin. For areas large enough for the average rainfall depth to depart considerably from that at a point, it has been found beneficial to convert point values to areal values. Frequency values for area-averaged precipitation are generally obtained by applying an areal correction factor (areal correction factor) to point precipitation values. There are many methods of transformation point values to areal estimates, with different results being obtained in the same network according to the method applied (Nguyen and others, 1981; Arnell and others, 1984; Niemzynowicz, 1982; Institute of Hydrology, 1999). The areal correction factor estimates depend on the raingauge network density and, consequently, on the accuracy of estimating the mean precipitation over an area. Most of the procedures that are used for computing mean areal precipitation from

raingauge data, such as the arithmetic average method, Thiessen polygon method and inversed distance-squared method, give comparable results for long time periods; but the differences in results among the various methods increase as the time period diminishes, as for daily rainfall. Dense networks of raingauges have been used to develop depth-area-duration correction factors (Smith, 1993; Institute of Hydrology, 1999). Areal correction factors depend on local climatological conditions and therefore, whenever possible, should be derived from local data. Validation is required if areal correction factors are to be used far from the location in which they were developed.

As procedures developed for converting point precipitation frequency values to areal values are mostly empirical, alternative methods have been proposed for directly carrying out areal precipitation frequency analyses using stochastic models of the spatial and temporal distributions of rainfall (Bras and Rodriguez-Iturbe, 1985; Smith, 1993).

5.7.5 Storm rainfall analysis for hydrological design applications

For design purposes, precipitation at a given site or over an area for a specified duration and return period is commonly used in the estimation of flood potential. The use of design precipitation to estimate floods is particularly valuable in those situations where flood records are not available or not long enough at the site of interest, or they are not homogeneous due to changes of watershed characteristics such as urbanization and channelization. Furthermore, design problems usually require information on very rare hydrological events: events with return periods much longer than 100 years. Common storm rainfall analysis techniques that can be used to address these design problems are discussed below.

5.7.5.1 Maximum observed rainfall

Some of the world's largest recorded rainfall amounts for selected durations are given in Table II.5.6. These values, which represent the current upper bounds on observed precipitation, are enveloped by the following approximate equation:

$$P = 422T^{0.475} \tag{5.33}$$

where P is the rainfall depth in millimetres, and T is the duration in hours. Most locations in the world will never come close to receiving these extreme rainfall amounts.

5.7.5.2 Rainfall intensity or depth–duration–frequency relationships

In standard engineering practice, the results of point-rainfall frequency analysis are often summarized by intensity–duration–frequency relationships or depth–duration–frequency relationships for each raingauge site with sufficient rainfall records. These relationships are commonly available in both tabular and graphical form for rainfall intensities or depths at time intervals ranging from five minutes to two days and for return periods from two to one hundred years. Owing to the uncertainties involved in extrapolation, rainfall values are generally not provided for return periods longer than roughly twice the raingauge record. Empirical equations expressing intensity–duration–frequency and depth–duration–frequency relationships have been developed. There are many such equations appearing in the technical literature, of which the following forms are the most typical:

$$i = \frac{a}{t^c + b} \tag{5.34}$$

$$i = \frac{aT}{t^c + b} \tag{5.35}$$

$$i = a(t - b)^{-c} \tag{5.36}$$

$$i = \frac{a + b \log T}{(1 + t)^c} \tag{5.37}$$

where i is the average rainfall intensity, that is, depth per unit time, generally expressed in mm/hr, t is the rainfall duration in minutes or hours, T is the return period in years, and a, b and c are coefficients varying with the location and return period.

5.7.5.3 Temporal and spatial extrapolation of point rainfall estimates

A number of publications (NRCC, 1989; ASCE, 1996; Pilgrim, 1998; Institute of Hydrology, 1999) provide mapped regional analysis of precipitation frequencies for various return periods and durations. For instance, the US Weather Bureau provides a rainfall atlas that contains maps for the entire United States with contour lines of rainfall amounts for durations varying from 30 minutes to 24 hours and return periods from 2 to 100 years (Hershfield, 1961). In addition to this atlas, the US National Weather Service has prepared isohyetal maps for rainfall events having durations from 5 to 60 minutes and for return periods of 2, 10, and 100 years for the eastern and central states (Frederick, and others, 1977). This set of maps is useful for

estimating design rainfalls of short duration or developing intensity–duration–frequency relationships.

Quantile estimates of point rainfall for durations and return periods not shown on the regional rainfall maps can be obtained by interpolation. For instance, for the eastern and central regions of the United States, depths for 10- and 30-minute durations for a given return period are computed by interpolation from the available 5-, 15- and 60-minute data for the same period (Frederick, and others, 1977):

$$P_{10min} = 0.41P_{5min} + 0.59P_{15min} \quad (5.38)$$

$$P_{30min} = 0.51P_{15min} + 0.49P_{60min} \quad (5.39)$$

For return periods other than 2 or 100 years, the following equations are used:

$$P_{Tyr} = aP_{2yr} + bP_{100yr} \quad (5.40)$$

in which *a* and *b* are empirical coefficients varying with return period values. Please note that these relationships are merely for illustration purposes. Owing to the regional variation in such a relationship, its application should be based on climatic similarity between the regions of its derivation and use.

In the absence of short-duration rainfall data, either at a site or sufficiently nearby for interpolation, it may be possible to estimate the rainfall regime from any indirect data that may be available. Such data include mean annual precipitation and mean annual number of days with rain, which may be obtained from maps or otherwise estimated. For the United States, the average relationship of precipitation per precipitation day (mean annual precipitation divided by days of precipitation with a base of one millimetre) to a 2-year 24-hour rainfall is as follows:

Precipitation per precipitation day (mm)	5	8	10	13
2-year 24-hour rainfall (mm)	36	56	79	107

Again, the relationship given in this table is merely for illustration. Owing to the regional variation in such a relationship, its application should be based on climatic similarity between the regions of its derivation and use.

For durations of less than 24 hours, it is advisable to estimate the 1-hour rainfall frequency amounts from the 24-hour values, to interpolate for intermediate durations and to extrapolate for durations one hour. The 2-year 1-hour rainfall is related to the 2-year 24-hour rainfall according to the mean annual number of days with thunderstorms. Studies that have included a wide range of climate indicate the following relationship:

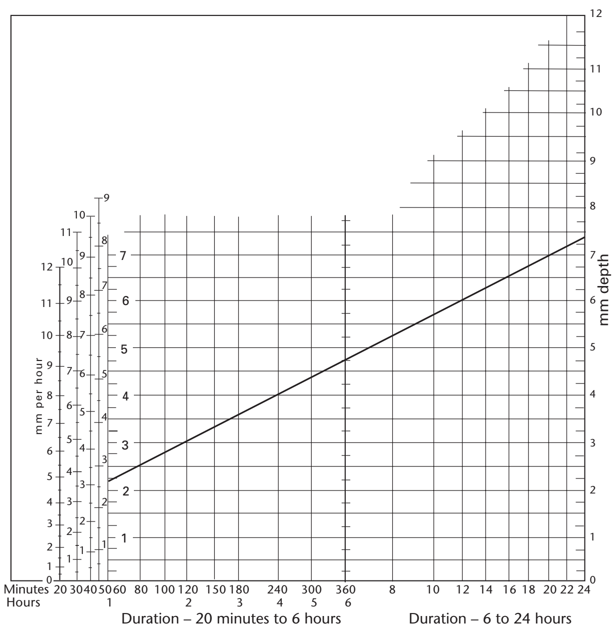
Ratio of 2-year 1-hour rainfall to 2-year 24-hour rainfall	0.2	0.3	0.4	0.5
Mean annual number of thunderstorm days	1	8	16	24

Rainfall-frequency values for durations of less than one hour are often obtained by indirect estimation. Rainfall data for such short durations are seldom readily available in convenient form for the compilation of annual or partial duration series for direct frequency analysis. Average ratios of rainfall amounts for 5, 10, 15 and 30 minutes to 1-hour amounts, computed from hundreds of station-years of records, are often used to estimate rainfall-frequency data for these short durations. These ratios, which have an average error of less than 10 per cent, are as follows:

Duration (minutes)	5	10	15	30
Ratio (n minutes to 60 minutes)	0.29	0.45	0.57	0.79

Thus, for example, if the 10-year 1-hour rainfall is 70 mm, the 10-year 15-minute rainfall is 57 per cent of 70, or 40 mm.

These ratios can yield erroneous results in some regions. For example, in regions where most of the rainfall occurs in connection with thunderstorms, the above ratios would tend to yield values that are too low. However, in regions where most of the rainfall results from orographic influences with little severe convective activity, the ratios might tend to yield values that are too high. This variation has been handled on a continental basis for Australia (Court, 1961; Hershfield, 1965), with a relationship that was developed by using a geographical location and 1-hour rainfall intensity as variables. The relationship is also dependant upon the average recurrence interval. When large quantities of rainfall data for a region are to be subjected to frequency analysis, as is usual in the preparation of generalized maps, the compilation of annual series data for



Note: For 20-minute to 60-minute rainfalls, values are in mm per hour;
for longer durations the values are in mm of depth.

Figure II.5.2. Rainfall–intensity and depth–duration relationship

all durations is a challenging and tedious task. It is customary, therefore, to limit such compilations to data from a relatively small number of recording stations with good records for at least ten years. The means of the annual series are then computed and used to prepare a diagram such as that given in Figure II.5.1, which permits the estimation of rainfall values for durations of up to 24 hours when the 1- and 24-hour amounts are known. The diagonal line in Figure II.5.2 illustrates an example in which 24-hour rainfall is about 73 mm and 1-hour rainfall is 22 mm. Values for other durations can be read off the intersections of the diagonal. Thus, the amount for 12 hours is 60 mm; for two hours it is 30 mm.

Diagrams similar to Figure II.5.3 may be constructed (Miller and others, 1973) for interpolating between the 2- and l00-year return periods. Such diagrams must be based on good long-record stations if they are to be reliable. As with the duration–interpolation diagrams, they vary from region to region, where climatic regimes differ significantly. They are used in the same manner as the duration–interpolation diagrams in that a diagonal is laid across the appropriate 2- and l00-year rainfall depths on their respective verticals, and depths for other return periods are read at the intersections of the diagonal with the corresponding verticals.

With the use of the above two types of interpolation diagrams, only the 1- and 24-hour rainfall amounts for the 2- and l00-year return periods need be computed for most of the stations in the region for which the diagrams were developed. The diagrams are then used to estimate other required values. Both types are subject to regional variations, and caution should be exercised in trying to apply the diagrams in regions other than those for which they were developed.

Another method for estimating extreme rainfall quantiles for locations without rainfall data is based on regional maps of rainfall statistics. For example, Environment Canada provides maps showing isolines of the mean and the standard deviation of annual rainfall extremes for each region of Canada for durations varying from 5 minutes to 24 hours (NRCC, 1989). Hence, if the Gumbel distribution is assumed to be acceptable for describing rainfall extreme distribution, the quantile estimate of rainfall for a given return period at an ungauged location can be computed using the frequency factor method and the corresponding interpolated values of rainfall statistics. Similarly, for Australia, under the assumption of log-normal and log-Pearson type III distributions for rainfall extremes, maps of regionalized skewness along with available rainfall frequency maps can be employed to derive intensity–duration–frequency curves for any given location using appropriate extrapolation and interpolation procedures (Pilgrim, 1998).

In summary, one of the main challenges for engineers and hydrologists is to obtain representative

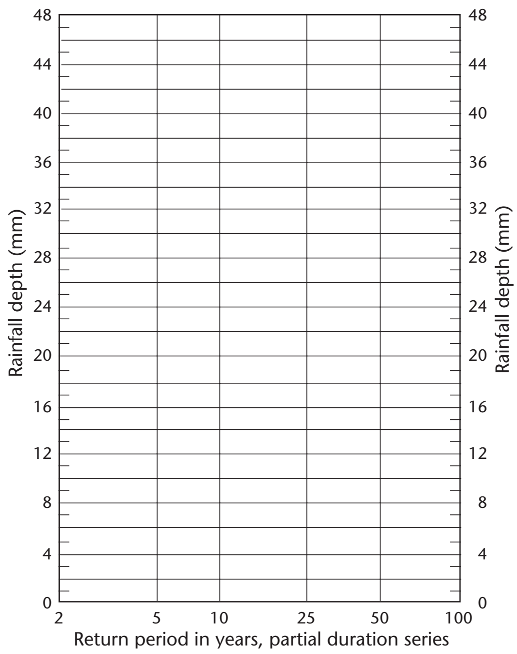


Figure II.5.3. Return-period interpolation diagram

information related to rainfall extremes at a given site. Precipitation stations, however, are not typically within close proximity to the site of interest, or they do not contain a sufficient period of rainfall records to allow a reliable estimation of rainfall. The rainfall frequency maps should be examined since they are sometimes based on the analysis of limited data over rather restricted areas, and the interpolation of rainfall characteristics to other areas could lead to grave uncertainties. Appropriate regional rainfall analysis procedures described in 5.7.3 should be used, especially for ungauged locations and for sites with limited rainfall records.

5.7.5.4 Mass rainfall curves

The first step in a storm-rainfall study is to plot accumulated values of rainfall versus time of day to give a mass curve, or integrated curve, for each station or for selected representative stations, if there are many. The mass curves for non-recording stations are constructed by comparison with mass curves from recording stations by means of proportionality factors. In doing so, the movement of the storm and any reports of the times of beginning, ending and heaviest rainfall should be taken into account. Figure II.5.4 shows a typical set of mass curves from the storm of 31 March–2 April 1962 in south-eastern Canada.

The pertinent stations are then listed in a table and accumulated values of rainfall are tabulated for each station for a pre-selected time increment. A 6-hour time increment is used in the present example, but other increments may serve equally well. For convenience, the stations should be listed in order of decreasing magnitude of total storm rainfall. The next step is to examine the table and select the particular 6-hour period that has the largest 6-hour rainfall increments. The values for this time increment are then listed. The period of maximum

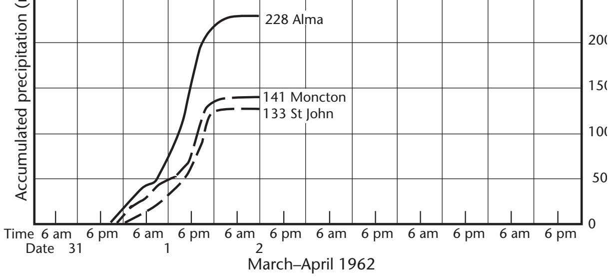


Figure II.5.4. Mass rainfall curves

Table II.5.7. Maximum average rainfall depth (mm) – storm of 31 March to 2 April 1962, south-eastern Canada

	Duration (hours)				
Area (km²)	*6*	*12*	*18*	*24*	*42*
25	90	165	205	230	240
100	85	155	190	215	225
1 000	70	130	165	185	190
10 000	50	90	115	140	145
100 000	25	45	65	75	85

12-hour rainfall is found in a similar way and its rainfall is listed. The same operation is applied to define the maximum 18-, 24-,..., n-hour increments. For periods embracing several 6-hour increments, a considerable number of trials may be required to find the period that includes the maximum rainfall for a particular duration.

5.7.5.5 Depth–area–duration analysis

Storm-rainfall analysis expresses the depth–area–duration characteristics of the rainfall from a particular storm. The depth is defined for pertinent combinations of enveloping area and duration, and is generally portrayed by tables or curves. In the aggregate, such analyses provide useful records for the design of flood control structures and for research in quantitative precipitation forecasting.

Individual point-rainfall observations are analysed jointly and in combination with other information. The rainfall data usually consist of observations of daily totals, interspersed with a few recorder measurements that contain short-term rainfall intensity information. Sometimes, these data are augmented by observations obtained through special interviews, referred to as bucket surveys. Additional information may come from synoptic weather maps, radar, reports of rises of small streams and other sources. The procedure, which is summarized in the following subsections, is described in detail in the *Manual for Depth–Area–Duration Analysis of Storm Precipitation* (WMO-No. 237).

Based on the tabulation of maximum rainfall increments, isohyetal maps are prepared for each duration, for example, 6 or 12 hours. Areas enclosed by each isohyet are then evaluated by using a planimeter or by tallying grid points, and the resulting values are plotted on a graph of area versus depth, with a smooth curve drawn for each

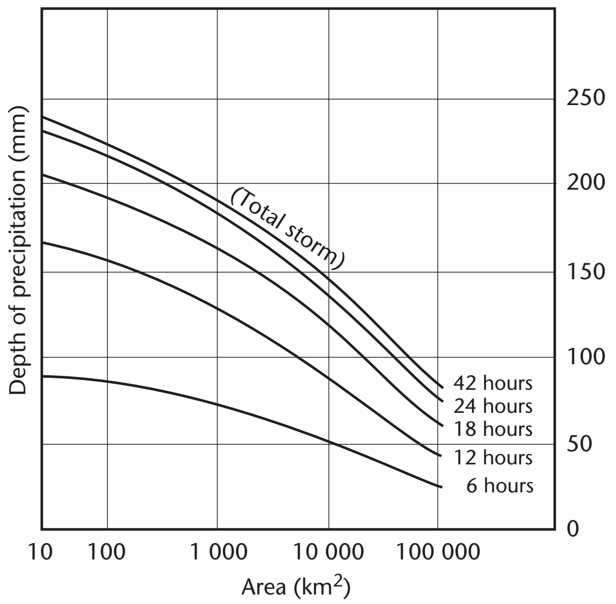


Figure II.5.5. Enveloping depth–area–duration curves

duration. A linear scale is commonly used for depth and a logarithmic scale for area. The enveloping or maximum depth–area–duration data for each increment of area and duration may be tabulated as in Table II.5.7 from curves such as those in Figure II.5.5.

5.7.5.6 Probable maximum precipitation

The term probable maximum precipitation, or PMP, is well established and is widely used to refer to the quantity of precipitation that approaches the physical upper limit of precipitation of a given duration over a particular basin. The terms maximum possible precipitation and extreme rainfall have been used with roughly the same meaning. To ask how possible or how probable such precipitation is would be at best a rhetorical question because the definition of probable maximum is an operational one that is specified by the operations performed on the data.

5.7.5.6.1 *Basic methods of estimating probable maximum precipitation*

There are two methods for estimating probable maximum precipitation: indirect and direct.

5.7.5.6.2 *Indirect type*

The indirect type first estimates probable maximum precipitation for the storm area, an area surrounded by isohyets, and then converts it into probable maximum precipitation for the design watershed. The main steps can be illustrated as follows:

High-efficiency storm → Moisture maximization → Transposition → Enveloping → Probable maximum precipitation → Probable maximum flood

High-efficiency storms are those for which the data support the assumption that their precipitation efficiency was near a maximum. The return period of such storms, given by point data on the enveloping curve, is usually more than 100 years.

Moisture maximization is a procedure by which the moisture of a high efficiency storm is maximized. The increase is usually limited to 20–40 per cent because there is an approximate physical upper limit for the representative dewpoint, which is a critical factor, and this cannot exceed the highest water temperature of the sea surface at the source of warm and wet air masses. In addition, this decreases as one moves from the source of air masses to the design watershed.

Transposition is a procedure which accounts for moving a high-efficiency storm from one location to another within a meteorologically homogeneous zone. In essence, it replaces time with space in order to increase the number of storm samples and provide additional observed data.

Enveloping refers to the use of a depth–area–duration relationship drawn up on the basis of transposed storms, thereby maximizing precipitation depths of various area sizes and durations. This also compensates for a lack of observed data.

5.7.5.6.3 *Direct type*

The direct type estimates probable maximum precipitation for the area directly encompassing the particular project in the design watershed. Major steps include the following:

Storm model → Maximization → Probable maximum precipitation → Probable maximum flood

The storm model for a typical storm or for an ideal storm reflects the characteristics of the catastrophic precipitation over the design watershed which is likely to pose the greatest threat of flooding for the project. Such models can be classified as local, transposition, combination or inferential, depending on their source.

Local models are used for local storm maximization and are selected from storm data observed in the design watershed. They can also be developed by simulating historically extraordinary floods from surveys.

Transposition models are derived by transposing actual storms in surrounding similar regions.

Combination models are sequences of two or more storms that are subject to spatial or temporal storm maximization and are combined in accordance with theories of synoptic meteorology.

Inferential models are theoretical or physical models which result from generalization and inference, using the three-dimensional spatial structure of storm weather systems within the design watershed whereby major physical factors that affect precipitation are expressed by a series of physical equations. They mainly include convergence models and laminar models of the flow field or wind field.

Maximization maximizes storm performance. When the storm model is that of a high-efficiency storm, then only moisture maximization is performed; otherwise both the moisture and power factors are maximized.

The above four methods are applicable to both hilly regions and plains. The fourth method is in general applicable to area of under 4 000 km^2 and durations shorter than 24 hours, whereas the other three methods are independent of area size and duration, and work especially well for estimating probable maximum precipitation for large watersheds larger than 50 000 km^2 and durations greater than three days.

Probable maximum precipitation can also be estimated by using the statistical estimation method and the empirical formula method.

5.7.5.6.4 *Preliminary considerations*

For major structures, the cost of the spillway may be a substantial proportion of the total project cost. Its design is therefore important enough to warrant a very detailed study. However, in the preliminary planning stages, it is sufficient to use generalized estimates of probable maximum precipitation if these are available for the area. Estimates of this type for the United States have been published as maps and diagrams in various issues of the US Weather Bureau Hydrometeorological Report series. Similar reports have been prepared by several other countries for various parts of the world. The following steps should be taken when determining probable maximum precipitation:

(a) Value basic data. Collect necessary hydrometeorological, geographic and orographic data, especially those related to extraordinary storms or floods and corresponding meteorological data, and asses their reliability;
(b) Make full use of storm data. Such data for the design watershed and its surrounding regions are the basis for calculating probable maximum precipitation and are also one of the major factors influencing the precision of results;
(c) Analyse characteristics and causes of large storms in the design watershed in order to provide a basis for determining methods for calculating probable maximum precipitation, selecting indicators, maximizing and analysing the reasonableness of results;
(d) Fully understand the characteristics of the methods. Select two or more methods from those that are available for determining probable maximum precipitation based on the conditions required for each method and the design requirements and data available for the watershed. Complete the calculation separately and then select the final results by means of a comprehensive evaluation.

5.7.5.6.5 *Requirements for probable maximum precipitation*

Unless depth–area–duration analyses applied to a project basin have been constructed within the storm-transposition zone, a number of individual storm studies will be required to obtain estimates of probable maximum rainfall. Before these studies are undertaken, the likely critical rainfall duration for the particular design problem should be determined. The selection of an appropriate tentative rainfall duration design can help avoid the analysis of data that are not directly applicable to the project and the subsequent need for analysis of additional data if too short a duration is adopted in the first instance.

The approximate time of rise of flood hydrographs for storms centring on different parts of the basin and the particular characteristics and proposed method of operation of the projected works should be considered in selecting tentative design rainfall duration.

The calculation undertaken should depend on the storm characteristics and design requirements of the project (Ministry of Water Resources and

Ministry of Energy of the People's Republic of China, 1995):

(a) If a project design requirement calls for probable maximum precipitation of a particular duration, only the storm volume and the most severe spatial or temporal distributions of that duration need be calculated;
(b) If the project calls for probable maximum precipitation of several durations, probable maximum precipitation should be determined for each of those durations.
(c) If the project involves a series of reaches along a river, as in a cascade of dams, then a series of probable maximum precipitation estimates will need to be made, with attention being paid to coordination between the upper and lower reaches. Regional estimates of probable maximum precipitation should be in accordance with the characteristics of observed storms;
(d) For places where storm characteristics differ among seasons, probable maximum precipitation estimates should be made for summer, autumn, rainy seasons, typhoons and so forth.

5.7.5.6.6 *Selection of sub-basins*

For project sites with large drainage areas, it may be necessary to estimate the probable maximum rainfall for some sub-basins and then compound the resultant probable maximum flood hydrographs from these sub-basins. To avoid subsequent unnecessary or incomplete analyses of mean areal rainfall depths during the storm studies, the sub-basins for which flood hydrographs are required should be selected before storm analyses are started. The selection of sub-basins is influenced by the physical characteristics of the basin and the availability and locations of stream-gauging stations from which the sub-area flood hydrographs can be routed to the project site.

Three commonly used methods are summarized below: the storm transposition method, the generalized estimation method and the statistical estimation method.

5.7.5.6.7 *Storm transposition method*

The basic assumption of storm transposition is that the region where the storm occurred – the storm source – and the design region are similar in terms of geographic or orographic conditions and the synoptic causes of storms. As a result, the structure – temperature, air pressure, wind power and spatial or temporal distributions – of a transposed storm is expected to change little. It includes the two following assumptions:

(a) After transposition, the storm weather system and the relative position of the storm area change little;
(b) After transposition, spatial or temporal distributions – the hyetograph and the isohyets – of the storm also change little.

5.7.5.6.8 *Selection of transposed objects*

Analyses should first be performed on the basis of data on observed catastrophic intense rainfall or floods which were collected from the design watershed in order to understand the basic types of catastrophic rainfall or floods in the watershed and then identify the storm types corresponding to probable maximum flood, PMF, required by the design project. For example, if the event in question is a tropical cyclone (typhoon, hurricane) or a frontal storm, the transposed object should be selected from among tropical cyclone storms or frontal storms, respectively.

5.7.5.6.9 *Possibility of transposition*

This involves a study of whether the selected transposed object is likely to occur in the design watershed. There are three solutions:

(a) Identifying meteorologically homogenous zones;
(b) Setting transposition limits for a particular storm;
(c) Performing specific analyses on the design watershed and comparing the similarity between the design watershed and the region of the storm source in terms of climate, weather, geography, orography and the like. The more similar these are, the more possible the transposition.

5.7.5.6.10 *Isohyetal map allocation*

Isohyetal map allocation moves the isohyetal map of the transposed object to the design watershed, which raises questions such as where to put the storm centre, whether to rotate the direction of the storm axis – the direction of the major axis of the isohyetal map – and how to rotate it.

The computations start with a study of the statistics of the spatial distribution of actual storms, that is, finding common rules of central positions and directions of axes of storms with weather causes similar to those of the transposed object on the basis of existing storm data, including those observed, surveyed and recorded in the literature, and then making adjustments and decisions in relation to the particular circumstances of the project.

The transposed isohyets should suit the large-scale orography of the design watershed as well as possible. The storm centre should match the small-scale orography, such as that surrounding the river channel.

5.7.5.6.11 *Transposition correction*

The purpose of transposition correction is to estimate quantitative changes to rainfall caused by differences in conditions such as geometry, geography and orography of the region. In other words, transposition correction typically includes the geometric, geographic and orographic corrections of the watershed. The geographic correction considers moisture correction only, while the orographic correction includes moisture correction and power correction. The geometric correction of the watershed must be performed first for any storm transposition.

If the transposed object is very similar to the design watershed with regard to the weather situation, orographic and geographic conditions are almost the same and there is no obvious moisture obstacle in between, the storm isolines of the transposed object may be moved to the design watershed without any change. Only a geometric correction of the watershed is needed.

If the two places are similar in terms of storm weather situation and different in terms of orographic and geographic conditions, and such differences are not large enough to cause great changes to the storm mechanism, then power correction need not be considered. In this case, only moisture correction needs to be considered in addition to the geometric correction of the watershed. This method is commonly used in plains and regions of low relief.

If storms with different orographic conditions must be transposed because of actual conditions, mountains will have some effects on the storm mechanism. In such cases, power correction needs to be considered in addition to geometric and moisture corrections of the watershed.

Concern for the orientation of precipitation patterns relative to basin orientations has resulted in special studies (WMO, 1986*a*; Hansen and others, 1982).

5.7.5.6.12 *Storm maximization*

In storm transposition, selected transposed objects are typically high-efficiency storms; therefore, only moisture maximization is needed when maximizing them. For such cases, maximization may be performed at the storm source before transposition only. Only after transposition correction, is the storm probable maximum precipitation.

Maximization methods developed in the United States and adopted in a number of countries (Pilgrim, 1998) have been described by Weisner (1970) and in a number of publications of the US National Weather Service, formerly the US Weather Bureau (see references in the *Manual for Estimation of Probable Maximum Precipitation* (WMO-No. 332, 1986*a*).

5.7.5.6.13 *Generalized estimation method*

This method involves estimating probable maximum precipitation for non-orographic regions and orographic regions respectively. It is assumed that precipitation in non-orographic regions results from the passing of weather systems, while that in orographic regions results from both the passing of weather systems and orographic effects. Precipitation caused by weather systems is referred to as convergence rain, or convergence components, and those caused by orography are called orographic rains, or orographic components.

Precipitation generalization involves the generalization of convergence rains, using the depth–area–duration generalization of storms. This generalization method is applicable to both a particular watershed and a large region that includes a lot of watersheds of various sizes. For the latter, it is called generalized or regional estimation. The content of generalization includes generalization of the depth–area–duration relationship and the generalization of the spatial/temporal distributions of probable maximum precipitation.

Determining probable maximum precipitation using the depth–area–duration generalized estimation method includes four steps:

(a) Maximize actual large storms – only moisture maximization being performed in most cases;
(b) Transpose maximized storms to the study region;
(c) Smoothen and fit envelope curves to data, including depth-duration smoothing, depth-area smoothing and combined depth–area–duration smoothing;
(d) Apply the probable maximum rainfall on the storm area to the design watershed so as to determine the probable maximum storm on the watershed area.

For regional generalized estimation, regional smoothing should be added after step (c). A check for regional consistency involving numerous comparisons has been described by Hansen and others (1977) and in the *Manual for Estimation of Probable Maximum Precipitation* (WMO-No. 332).

The method is used to estimate probable maximum precipitation for durations of 6 to 72 hours and for areas under 52 000 km^2 in plains and areas under 13 000 km^2 in orographic regions in the United States. For orographic regions, the influence of the topography should be considered in probable maximum precipitation estimation. For other countries or regions, the area sizes to which the method is applicable need to be analysed, based on the actual local conditions.

The method makes full use of maxima, including the largest rainfalls for various durations and areas of all the storm data in the particular region. The results of these calculations can be coordinated in the region and the watershed.

Now widely used in the United States, Australia, India and other countries, the generalized estimation method is described in the *Manual for Estimation of Probable Maximum Precipitation* (WMO-No. 332).

Major results of the generalized estimation method include the following:

(a) The precipitation depth of probable maximum precipitation: one is the enveloping curve map of the depth–area–duration relationship of probable maximum precipitation and the other is the probable maximum precipitation isoline map for several durations and area sizes;
(b) The spatial distribution of probable maximum precipitation: generalized as a set of concentric, similar ellipses;
(c) The temporal distribution of probable maximum precipitation: generalized as a single peak;
(d) For orographic regions, there are also some correlograms or isoline maps of some parameters that reflect orographic effects, which are used to convert probable maximum precipitation of convergence rains into probable maximum precipitation for orographic regions.

5.7.5.6.14 *Statistical estimation method*

This is an approximate method for estimating probable maximum precipitation for small watersheds, usually those under 1 000 km^2. It is summarized below.

In principle, probable maximum precipitation for small watersheds may be determined using the storm transposition method. Nonetheless, when the design region lacks the moisture and wind data needed for maximization, it will be very hard to use the traditional storm transposition method. If an abstracted statistical value K_m is transposed instead of transposing directly the rainfall of a storm, the issue will be much simpler. K_m may be defined by:

$$K_m = \frac{X_m - \bar{X}_{n-1}}{S_{n-1}} \qquad (5.41)$$

where X_m is the first item in the ranked observed series, that is, the very large value,

$\bar{X}_{n-1}$ is the average excluding the very large value, that is:

$$\bar{X}_{n-1} = \frac{1}{n-1}\sum_{i=2}^{n} X_i \qquad (5.42)$$

S_{n-1} is the standard deviation excluding the very large value, that is:

$$S_{n-1} = \sqrt{\frac{1}{n-2}\sum_{i=2}^{n}(X_i - \bar{X}_{i-1})} \qquad (5.43)$$

Clearly, the more data that are used and the more regions that are studied, then the enveloping value of K_m will be closer to the value corresponding to probable maximum precipitation.

Hershfield (1965) collected data from more than 2 600 rainfall stations, about 90 per cent of which were in the United States and developed a graphical relationship between enveloping values and means of annual series of K_m for different durations (Figure II.5.6) for the use of hydrologists.

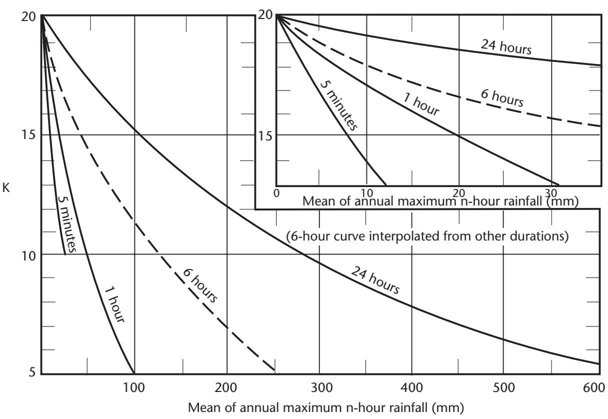


Figure II.5.6. K as a function of rainfall duration and mean of annual series

When using Figure II.5.6 to determine K_m, the average $\bar{X}_n$ and S_n are worked out based on rainfall data from a particular station in the design watershed and the calculation is completed according to the following equation:

$$PMP = \bar{X}_n + K_m S_n \tag{5.44}$$

The coefficient of variability is:

$$C_{vn} = \frac{S_n}{X_n} \tag{5.45}$$

Therefore, equation 5.39 can be rewritten as follows:

$$PMP = (1 + K_m C_{vn})\, \bar{X}_n \tag{5.46}$$

As illustrated by equation 5.46, determining probable maximum precipitation with Hershfield's statistical estimation method is essentially a matter of transposing the statistical value K_m of a very large storm in a wide region and correcting it by using the storm average $\bar{X}_n$ and the coefficient of variability C_{vn} for the design watershed. The method requires that enough single-station, daily precipitation observation series be available for the design watershed.

Maximum rainfalls needed are selected from among records using a particular duration or durations (1 hour, 6 hours, 24 hours) each year and are organized into an annual series. The mean $\bar{X}$ and the standard deviation S_n or the coefficient C_{vn} of the series are then calculated. The K value is determined from Figure II.5.6 using the mean of the series. As a result, probable maximum precipitation can be determined according to equation 5.44 or 5.46.

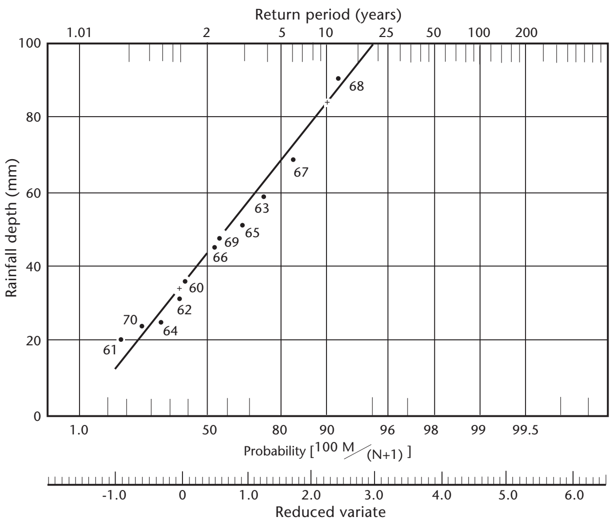


Figure II.5.7. Example of an extreme probability plot

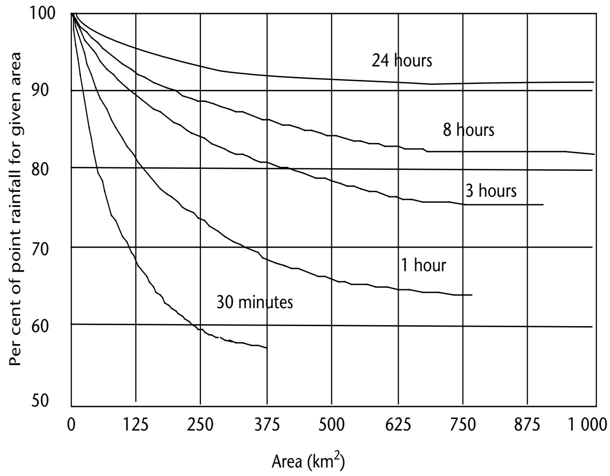


Figure II.5.8. Depth–area curves

Care should be taken to ensure that the highest one or two values in the annual series are consistent with the other values comprising the series. If, for example, the maximum value in a 30-year period is twice the second-highest value, it is clearly an outlier. The easiest way to detect an outlier is to arrange the series in descending order and then compute the return period of each value. Next, the values are plotted against their corresponding return periods on probability paper as shown in Figure II.5.7. If the maximum value of the series lies well above the line delineated by the other items in the series, it can be considered an outlier. An outlier should not be used to compute the mean or standard deviation of the series. If used, the mean and standard deviation should be adjusted as indicated by Hershfield, who also provided an adjustment for length of record. A complete, detailed description of the entire procedure, including diagrams for making the necessary adjustments, is given in the *Manual for Estimation of Probable Maximum Precipitation* (WMO-No. 332), Chapter 4.

When the probable maximum precipitation is to be applied to an area larger than about 25 km², it should be reduced. No modification is considered necessary for smaller areas. For larger areas, the point value is generally reduced by means of depth area or area reduction curves similar to those of Figure II.5.8.

The statistical method described above may overestimate the probable maximum precipitation in regions of heavy rainfall and in regions of frequent storms of similar types. In regions of low rainfall

and where heavy rain-producing storms, such as tropical cyclones, are rare but possible, the method may underestimate probable maximum precipitation. Values of K as high as 30 have been found necessary in order to exceed maximum observed point rainfall amounts in some regions. In some countries, in particular the United States, where storm studies are the preferred source of data for probable maximum precipitation determination, the statistical method has been used primarily as a means of checking for consistency.

5.7.5.6.15 *Checking the plausability of probable maximum precipitation estimates*

In principle, a variety of methods should be used concurrently to estimate probable maximum precipitation. Results of those methods should then be analysed comprehensively to select the best probable maximum precipitation value. In the end, the plausibility of the selected probable maximum precipitation should be checked from multiple perspectives so that the result is both maximal and possible. In general terms, methods of checking the rationality of probable maximum precipitation results are the same as those for the plausibility of probable maximum flood results. As a result, methods for checking them are the same (see 5.10.2 or *Manual for Estimation of Probable Maximum Precipitation* (WMO-No. 332), Chapter 4.

5.7.5.7 Design storm

A design storm or design hyetograph is a rainfall temporal pattern that is defined for used in the design of a hydraulic structure. A design hyetograph or synthetic storm of specified exceedance probability can be developed in the following way. The rainfall depth is obtained from the depth–duration–frequency relationship based on the specified probability and duration. Next, an area adjustment factor is applied to the rainfall depth. Finally, a method is used to distribute the rainfall depth over time using available procedures (Wenzel, 1982; Arnell and others, 1984). Pilgrim and Cordery (1975) warn that approaches overly smoothing the temporal patterns of rainfall are unsuited for design applications because the time variability of rainfall intensity often has a significant effect on the design hydrograph. Two important points noted by Pilgrim and Cordery (1975) and Huff and Changnon (1964) are that the variability of intensities diminishes with decreasing exceedance probability and the majority of extreme storms have multiple peaks of high rainfall intensity. Depth–duration–frequency relationships can be regionalized using procedures described above.

5.7.5.8 Drought

Drought is the low hydrological extreme resulting from perturbations in the hydrologic cycle over a sufficiently long time to result in a significant water deficit. Local water resources become insufficient to support the established or normal activities of the area. Droughts are interpreted and categorized broadly as meteorological, hydrological or agricultural. The meteorologist is concerned with drought in the context of a period of below-normal precipitation. To a hydrologist, drought refers to below-average flow in streams or content in reservoirs, lakes, tanks, aquifers and soil moisture. To an agriculturist, drought means a prolonged shortage of soil moisture in the root zone.

For meteorological drought, a useful means of analysis is based on the magnitude-span frequency. A simple type of analysis would compare rainfall totals for calendar months or pertinent seasons with their corresponding normal values and assess severity of drought based on negative departures from normal values. To take into account the effect of time distribution of rainfall, an antecedent-precipitation index may be used instead of total rainfall. Another way to account for the month-to-month carry-over effects of rainfall for evaluating severity of meteorological drought is the Herbst technique (Herbst and others, 1966).

The severity of agricultural drought may be judged by the drought index, a means of summarizing and periodically disseminating drought information and crop-moisture conditions on a regional basis. It can be used for evaluating the drought hazard over a sizeable area or for periodic assessment of the current extent and severity over a region.

Hydrological drought severity is related to the severity of departure from the norm of low flows and soil moisture in conjunction with excessive lowering of groundwater levels. In view of the considerable time lag between departures of precipitation and the point at which these deficiencies become evident in surface water and groundwater, hydrological drought is even further removed from the precipitation deficiency since it is normally defined by the departure of surface and subsurface water supplies from some average condition at various points in time.

5.7.5.9 Recent precipitation frequency analysis techniques

The density of raingauges has been a significant limitation in the development of precipitation frequency analysis procedures. Radar provides a

potentially important source of precipitation data for frequency analyses. The most important advantage of radar for precipitation measurement is the coverage radar provides of a large area with good spatial and temporal resolutions, as small as 1 km^2 and 5 minutes. With an effective range of 200 km, a single radar can cover an area of more than 10 000 km^2.

Cluckie and others (1987) report a depth–area–duration analysis of extreme events using hourly radar rainfall totals for five-km grid squares. The need to first correct and calibrate radar data is emphasized. Depth–area–duration analysis is performed on individual storms as a means of classifying their flood producing potential. Furthermore, Cluckie and Pessoa (1990) have used radar data from northwest England to characterize actual storms, which have then been maximized and transposed to obtain probable maximum precipitation estimates for catchments areas of interest (see 5.7.5.6 for a discussion of probable maximum precipitation). Such an approach capitalizes on radar's ability to delineate storms in space and time. In addition, a program called RADMAX implements the procedure and incorporates visualization of the storm transposition step (Moore, 1993). Collier (1993) suggested the use of radar, and satellite data for cruder estimates to support probable maximum precipitation estimation by using the storm model approach.

Design problems generally require information on very rare hydrological events, namely those with return periods much longer than 100 years. Traditional techniques for addressing these design problems are mostly based on the use of probable maximum precipitation. New frequency analysis procedures, which exploit some of the tools of probable maximum precipitation, have been developed for assessing rainfall magnitudes with very long return periods. In particular, the National Research Council (1988) recommended the stochastic storm transposition techniques. In the probable maximum precipitation application, storm transposition is based on the assumption that there exist meteorologically homogeneous regions such that a major storm occurring somewhere in the region could occur anywhere else in the region, with the provision that there may be differences in the averaged depth of rainfall based on differences in moisture potential. In the stochastic storm transposition method, the frequency of occurrence of storms in the transposition region provides the link for obtaining frequency estimates of extreme storm magnitudes. The stochastic storm transposition provides estimates of the annual exceedance probability of the average storm depth over the catchment of interest. The estimate is based on regionalized storm characteristics such as maximum storm centre depth, storm shape parameters, storm orientation, storm depth and spatial variability, and on an estimation of the joint probability distribution of storm characteristics and storm occurrence within a region. An advantage of the stochastic storm transposition method is that it explicitly considers the morphology of the storms, including the spatial distribution of storm depth and its relation to the size and shape of the catchment of interest (NRC, 1988).

5.8 LOW-FLOW ANALYSES [HOMS I80, K10]

5.8.1 General

Information on the characteristics of low flows for streams and rivers is important for planning, design and operation of water-related projects and water resource systems. Such information is used in designing wastewater treatment and storage facilities to ensure that releases do not exceed the assimilative capacity of receiving waterways, reservoir storage design for multi-purpose systems and the allocation of water for various purposes such as industrial, agricultural, domestic and in-stream ecological needs.

Low-flow frequency analysis and flow-duration curves are the two most commonly used analytical tools to help assess the low-flow characteristics of streams, and these will be described in more detail in this section. Both approaches typically require at-site continuous streamflow data for analysis, unless regional approaches are used to estimate at-site characteristics. Other characteristics that are sometimes useful include the amount of time or frequency for which flows might be below a certain threshold during a season and the volume of water or deficit that might arise during the period in which flows are below a threshold. Statistical approaches can also be used to assess these aspects. Other approaches, such as passing historical sequences of data or synthetically generated sequences through a model of the river or reservoir system can provide additional, valuable detailed information for design purposes. The latter approaches will not be covered in this Guide.

Low flows are usually sustained by depletion of groundwater reserves or by surface discharge from upstream bodies of water including lakes, wetlands

and glaciers. Low flows within a year or season may result from different mechanisms forcing the hydrological response. Low flows in cold, northern climates may occur due to the prolonged winter period where precipitation occurring during this period is primarily in the form of snow, resulting in ever-decreasing flows until the occurrence of the spring freshet. A second period that produces low flows occurs during the warm season where there may be periods of significant evaporation and little precipitation. Depending on local climatology and physiography, some basins may produce low flows resulting predominately from one process or a combination of processes as described above (Waylen and Woo, 1987). It is important to understand the processes producing the low flows, as these may determine the analytical approaches taken to analyse their characteristics and results.

Anthropogenic intervention can greatly alter the natural low-flow regime. For example, increased extraction from surface water for irrigation may occur during periods of prolonged absence of rainfall, resulting in artificially suppressed flow values, compared with what naturally would have occurred. Significant extraction of groundwater for agricultural, industrial and human uses can reduce water-table levels and result in reduced streamflow. A variety of other anthropogenic interventions can occur within a basin and should be known prior to proceeding with analyses of data. Such interventions can include upstream regulation, inter-basin transfers, return flows from domestic sewage systems that use groundwater as a water source and changes in land use, such as deforestation, reforestation and urbanization. Such operations may cause increases or decreases in flow rates (Institute of Hydrology, 1980; Smakhtin, 2001) and may well invalidate the assumptions commonly associated with the analytical tools described below and in previous sections of this chapter.

5.8.2 At-site low-flow frequency analysis

Information on low-flow frequency is obtained from an analysis relating the probability of exceeding an event to its magnitude. Such relationships can be established for low flows of various durations, such as 1, 3, 7 or 14 days or other periods or durations of interest. Commonly, non-parametric frequency analysis or probability distributions are used to describe the frequency relationship of observed seasonal or annual low flows. As in the case of flood flows, the parent distribution of low flows is unknown.

Various studies have been conducted to ascertain which distributions and which parameter estimation methods may best represent the distribution of low flows (see for example Nathan and McMahon, 1990; Lawal and Watt, 1996; and Durrans and Tomic, 2001). Results of the studies tend to differ, as the same distributions, fitting methods or data are not always used.

Matalas (1963) analysed data for 34 sites in the United States using the Pearson type III (P3), the Pearson type V (P5), the Gumbel type III (G3), which is also known as the three-parameter Weibull (W3), and the three-parameter log-normal (LN3) distributions. He concluded that the G3 and P3 distributions performed equally well and tended to outperform the other two distributions. According to Matalas (1963), the theoretical probability distribution should have a lower boundary greater than or equal to zero, and he used this as one criterion in assessing the acceptability of a distribution. Condie and Nix (1975) performed a similar analysis of data from 38 Canadian rivers using the same probability distributions as Matalas (1963). To ascertain the suitability of the distribution, they considered solutions in which the lower boundary parameter was greater than zero and smaller than the smallest observed flow. They recommended the use of the G3 distribution, with parameters estimated by maximum likelihood, followed by the method of smallest observed drought. Condie and Cheng (1982), furthering the work of Condie and Nix (1975), continued to recommend the use of the G3 distribution for low-flow frequency analysis. In the latter study, they considered a negative lower boundary to be acceptable. In such cases, they took the area of the density function from the negative lower boundary to zero as representing the probability of the occurrence of zero flows. They also verified that the lower boundary parameter was not larger than the smallest member of the sample, as certain fitting methods can provide such unrealistic results.

Tasker (1987) showed that for 20 stations in Virginia, United States, using bootstrapping that the log-Pearson type III (LP3) and G3 distributions had lower mean square errors in estimating the 7-day 10-year (Q7,10) and 7-day 20-year (Q7,20) low flows than did the Box–Cox transformations or the log-Boughton methods. Vogel and Kroll (1989) analysed the two-parameter log-normal (LN2) and two-parameter Weibull (W2) distributions fitted to data from 23 sites in Massachusetts, United States. They concluded that the W2 distribution fitted poorly, while there was no evidence to reject the hypothesis that the data were from a LN2 distribution. In addition, they analysed a variety of three-parameter

distributions, namely the LN3, the LP3 and the G3. They found that the LP3 slightly outperformed the other three- and two-parameter distributions. These studies indicate that the preferred frequency distribution varies by region and there is no one frequency distribution that clearly outperforms all others.

Zaidman and others (2003) performed an analysis of 25 natural streams within the United Kingdom having more than 30 years of record. They derived data times series for durations of 1, 7, 30, 60, 90 and 365 days for each of the basins. In turn, four three-parameter distributions, namely the generalized extreme value distribution, generalized logistic distribution (GL), P3, and generalized Pareto distribution were used to fit the data for each of the series and for each duration. Goodness-of-fit tests and judgment were used to discern the results. The findings were as follows:

(a) The candidate distributions fit the observed data points very well, with little quantitative evidence to differentiate between them;
(b) Certain distributions performed better than others with the distribution type varying with duration and basin characteristics;
(c) The base flow index (Institute of Hydrology, 1980) was very useful to quantify basin geology;
(d) With regard to less permeable basins, the P3 provided the best results for shorter durations, with the generalized extreme value surpassing the P3 for longer durations;
(e) For more permeable basins, the GL provided the best results.

It has been commonly observed (Nathan and McMahon, 1990; Durrans and Tomic, 2001) that the largest flows within a series of minima are often described more effectively by a much steeper cumulative distribution curve than would be used to describe the subsequent lower flows. In response to this phenomenon, approaches have been developed to fit only the lower portion or tail of the distribution, rather than fitting the distribution to the entire sample. Nathan and McMahon (1990) noted that a transition seems to occur where the “higher frequency flows are no longer considered” as low flows but represent more “normal conditions”. Approaches such as conditional probability adjustment (Condie and Cheng, 1982; Nathan and McMahon, 1990), the application of censoring theory (Kroll and Stedinger, 1996), mixture or compound parametric models (Waylen and Woo, 1987) and non-parametric frequency approaches (Adamowski, 1996; Guo and others, 1996) have been advocated to compensate for sample heterogeneity. Such approaches can also be used to perform an analysis when zero flow values are present in the sample.

Durrans and Tomic (2001) explored the performance of a number of methods that place an emphasis on fitting only the tails of the distributions. They concluded that the various methods performed “about as well as, if not better than, an estimation strategy involving fitting” the entire dataset to the LN distribution using L–moments. In contrast to this approach, for areas where annual or seasonal low-flow series may be generated by more than one mechanism and if these mechanisms can be identified, a mixture or compound parametric model could provide a more reasonable description of the data. Alternatively, non-parametric frequency estimation, as proposed by Adamowski (1996) and Guo and others (1996), could be employed. Furthermore, it has been shown that non-parametric estimation procedures provide estimates of low-flow quantiles as accurate as or more accurate than those produced by more traditional parametric approaches, namely the LP3, W2 and W3 distributions, based on simulation experiments with homogenous samples.

Low-flow statistics are generally computed for periods or durations of prescribed lengths, such as 1, 3, 7, 14, 30, 60, 90, 120, 183 and 365 days. The low-flow data for various durations are computed using a moving average for the desired period. The moving average is the lowest arithmetically averaged flow of d consecutive days within a given year. As a rule, these values are computed over a hydrological or climatic year rather than a calendar year. The hydrological year is defined to start in a season when the flow is most likely to be high so that yearly low-flow periods are not likely to be partitioned into different years. Statistics such as the mean annual d-day minimum can be computed, as can the d-day, T-year low-flow statistic, commonly denoted as Qd,T. In general, the specific d-day duration is selected according to agricultural, biological or engineering applications, which are usually related to the impact of the risk associated with the duration of low water availability on the system under study. Computational methods for estimating the parameters of the distribution of the d-day series are similar to the methods described for flood frequency analysis, with some minor variations, such as the parameter estimation method of smallest observed drought for the G3 distribution.

Two HOMS components are of particular interest for estimating low-flow frequency statistics of d-day durations. They are I80.2.03, the low-flow frequency analysis package, which allows testing of the hypotheses for randomness, homogeneity, trend

and independence, and I80.2.04, Program LOWSTATS, the low-flow statistical package.

Limited analyses have been performed for durations in excess of one year and the frequency of these multi-year flows have been determined using plotting positions (Carswell and Bond, 1980; Paulson and others, 1991). Frequency analyses of multi-year low flows are important in water-supply storage analysis where carry-over storage from year to year is required to meet water-supply demands. HOMS component I80.2.05 Program DROUGHT, estimation of the probability of occurrence of n-month droughts, can be used to facilitate the analysis of multi-year events.

Examples of low-flow frequency curves for various durations are shown in Figure II.5.9. The low-flow data are typically plotted with a logarithmic or arithmetic scale for the ordinate and a normal probability scale or Gumbel scale as the abscissa. Although few data samples will plot as a perfect straight line, these types of paperare used to visually assess the overall fit of the model to the data. Special graph paper has been constructed to allow the normal and Gumbel distribution to be drawn as a straight line. Methods have also been developed to change the scale of the abscissa for various three-parameter distributions such that the cumulative distribution function will plot as a straight line (Vogel and Kroll, 1989). This change of scale would be valid for only one curve within the family of curves for the particular family of distributions, as the skewness would most likely change with duration. The technique of adjusting the abscissa to reflect sample skewness is not commonly employed in the graphical representation of low-flow results.

5.8.3 Low-flow frequency estimation at partial-record sites using base-flow measurements

The methods described thus far are valid for gauged sites having sufficient data upon which to base a

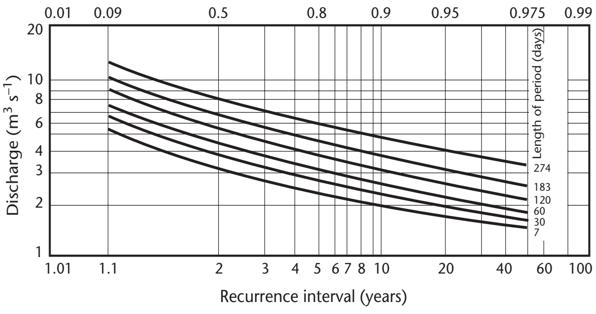


Figure II.5.9. Frequency curves of annual low flow

frequency analysis: usually 10 years or more. However, discharge measurements made at ungauged sites during times of low or base flow can be used in conjunction with concurrent daily flows at nearby gauged sites to estimate low-flow frequency. Sites where only base-flow measurements are available are referred to as partial-record sites. A relation is established between the base-flow measurements at the partial-record site and concurrent daily flows at the nearby gauged site. This relation and low-flow characteristics at the gauged site are used to estimate d-day, T-year flows at the partial-record site. The gauged site should have topographic, climatic and geological characteristics similar to the partial-record site. In order to achieve a linear relation, the logarithms of concurrent base-flow measurements $\tilde{y}$, at the partial-record site, and daily flows $\tilde{x}$, at the gauged site, are normally used to estimate the parameters of the linear relation. Such observations should be separated by significant storm events so as to represent reasonably independent observations of the low-flow processes. At least 10 paired observations are needed to define the relation between concurrent base-flow measurements $\tilde{y}$, and the daily flows $\tilde{x}$. The analysis is based on the assumption or approximation that the relation between $\tilde{y}$ and $\tilde{x}$ can be described by:

$$\tilde{y} = a + b\tilde{x} + e \quad e \sim N(0,s_e^2) \tag{5.47}$$

Where: a, b, and s_e^2 are the constant, coefficient and variance, respectively, of the linear regression equation. It is assumed that the residuals, e, are independent and normally distributed. The estimators of the mean, $M(y)$, and variance, $S^2(y)$, of the annual minimum d-day low flows at the partial-record site are as follows:

$$M(y) = a + b\,M(x) \tag{5.48}$$

and

$$S^2(y) = b^2\,S^2(x) + S_e^2[1 - (S^2(x)/(L-1)S^2(\tilde{x}))] \tag{5.49}$$

where $M(x)$ and $S^2(x)$ are the estimators of the mean and variance of the annual minimum d-day low flows at the gauged site, L is the number of base-flow measurement and $S^2(\tilde{x})$ is the variance of the concurrent daily flows at the gauged site.

The primary assumption is that the relationship between instantaneous base flows, as shown in equation 5.47, is the same as the relation between the annual minimum d-day low flows at the two sites. This is a necessary assumption if the proposed method is to be used to estimate the

d-day, T-year low flows at the partial-record station. While this approximation appears reasonable for d-day means up to about seven days, it may not be satisfactory for durations significantly longer than this. Stedinger and Thomas (1985), and Thomas and Stedinger (1991) discuss the use of base-flow measurements to estimate low-flow characteristics at partial-record sites in the United States.

The d-day, T-year low flow at the partial-record site is estimated using the mean and variance given in equations 5.48 and 5.49. If a three-parameter distribution is used to estimate the d-day, T-year flow at the partial record site, then the skewness of the gauged site is assumed to be the same that at the partial-record site. As described earlier, the d-day, T-year low flow at the gauged site can be estimated using procedures described in 5.8.2. Stedinger and Thomas (1985) explain why the d-day, T-year low flow at the gauged site cannot simply be used as the independent variable in equation 5.47. A loss of variance is associated with using least-squares regression equations to estimate frequency estimates such as the d-day, T-year low flow. In particular, substituting the d-day, T-year low flow at the gauged site in equation 5.47 would tend to overestimate the d-day, T-year low flow at the partial-record site. Stedinger and Thomas (1985) developed a procedure for obtaining an unbiased estimate of the variance of the annual d-day flows at the partial-record site using the relation in equation 5.49 and the variances of the annual d-day low flows and the concurrent daily flows at the gauged site.

Stedinger and Thomas (1985) also developed a procedure for estimating the standard error of the d-day, T-year low flow at partial-record stations. They illustrate that the standard error is a function of the correlation between the base-flow measurements and daily flows, the number of base-flow measurements made at the partial-record site, the magnitude of the departure of the d-day, T-year low flow at the gauged site from the mean of the daily flows used in equation 5.43 and the record length at the gauged site. Using data for 20 pairs of gauging stations in the eastern United States, Stedinger and Thomas (1985) illustrated that standard errors of about 30 percent can be achieved for partial-record stations when the correlation coefficients exceed about 0.7 and there are 15 base-flow measurements and 25 years of record at the gauged site. Using data for 1 300 gauging station in the continental United States, Reilly and Kroll (2003) demonstrated that the base-flow correlation approach gave improved results over regional regression models for 15 of the 18 major river basins in the United States. Because the method utilizes at-site data, the base-flow correlation method generally provides more accurate estimates of d-day, T-year low flows than would the regional regression models described in the next section.

5.8.4 Regionalization of low-flow frequency statistics

The methods described thus far are valid for sites having sufficient data upon which to base a frequency analysis or for which base flow measurements are available. Such sites should be relatively free of human intervention and should be of sufficient record length as to provide an accurate representation of low-flow statistics for the basin. These statistics can be estimated for ungauged basins based on regionalization methods or through the abstraction of statistics from generated time series data obtained through statistical or deterministic modeling. The first approach is most commonly used to estimate the low-flow statistic of interest, for example the seven-day, two-year low flow, Q7,2, at ungauged sites. The statistic of interest is regressed against a number of independent or explanatory variables. These independent variables represent physical and climatic characteristics of the basin. Such approaches have been used with success for the estimation of design floods, but it has been found to be much more difficult to find accurate regression models to estimate low-flow statistics (Vogel and Kroll, 1992; Waltemeyer, 2002).

Regionalization generally entails the identification of homogeneous regions over which a particular regression equation applies. Regionalization is an attempt to group basins geographically or in a multivariate space, which may not result in geographically contiguous regions, based on physiographic, climatic or streamflow characteristics. In generaly, the ability to define homogeneous regions results in increased predictive accuracy and more meaningful physical models for the statistical estimation procedure (Nathan and McMahon, 1992; Waltemeyer, 2002).

HOMS component K10.2.04, regional analyses of streamflow characteristics, describes approaches for developing regional relationships between streamflow and basin characteristics.

Regional low-flow models are generally expressed in the following form:

$$Q_{d,T} = aX_1^{\,b}X_2^{\,c}X_3^{\,d}\ldots \tag{5.50}$$

where $Q_{d,T}$ is the d-day, T-year low-flow statistic, the X_i are basin physiographic or climatic characteristics, and a, b, c and d are parameters obtained through multiple regression analysis (Weisberg, 1980; Draper and Smith, 1981). Various low-flow statistics are estimated from an at-site frequency analysis of the data from different sites within a region. Basin and climatic characteristics are, in turn, derived from maps or from climatological data (see Institute of Hydrology (1980), Vogel and Kroll (1992) and Waltemeyer (2002). The parameters of the equation can be estimated using ordinary, weighted or generalized least squares techniques. Although the technique of generalized least squares is more difficult to apply than ordinary least squares, Vogel and Kroll (1990) observed in their modeling of 23 basins in Massachusetts that the estimated parameters and the t-ratios obtained using the two approaches were almost identical. However, the more complex approach provides information on the composition of the error of prediction, allowing the attribution of error to model error, measurement error and sampling uncertainty. Vogel and Kroll (1990) noted that model error was by far the major component of the prediction error. Their analysis helps to emphasize the importance of establishing more physically meaningful statistically based model.

Regional regression equations of the form of equation 5.50 are applicable for regions where the d-day, T-year low flows are non-zero. Tasker (1991) has developed procedures for estimating low flows in regions where the d-day, T-year low flow may be zero. These techniques involve developing regional relationships with censored data and the use of logistic regression to estimate the probability of the d-day, T-year being zero.

Numerous basin and climatic characteristics have been used in regional regression equations to estimate low-flow statistics. Most models include a drainage area and a variable representing climatic conditions, such as mean annual precipitation. Many other characteristics have been considered, some of which are the mean watershed elevation, proportion of basin in forest cover, proportion of basin in lakes and swamps, average basin slope, drainage density, main channel slope and proportion of urban area. Given that low flows are normally a result of the prolonged absence of rainfall, it is commonly thought that their low-flow characteristics should be closely related to the underlying geological and soil characteristics of the basin (Institute of Hydrology, 1980; Task Committee of the Hydraulics Division, 1980).

In certain cases, improved relationships have been attained by including an explanatory variable representing a geological index. Such indexes have seen increasing popularity and have led to increases in model performance. The base flow index (Institute of Hydrology, 1980) could be considered to reflect, in part, basin geology and is the ratio of flow, generally known as baseflow, to the total flow. According to Gustard and Irving (1994), a soil index can lead to improved prediction models.

Another approach has been taken to improve linkages between low-flow characteristics and recession curves or coefficients for the basin. Bingham (1982) defined a streamflow recession index, in days per log cycle of discharge depletion, at gauged streams in Alabama and Tennessee (United States) and then mapped the index according to computed indices at gauging stations and a geological map for use in estimating low-flow characteristics for ungauged streams. Vogel and Kroll (1992) formulated a conceptual model of the form of equation 5.50 to relate the unregulated flow of a basin during recession periods to the basin's characteristics. They regressed Q7,10 with three of the five variables of the conceptual model. Vogel and Kroll found dramatic increases in accuracy by inclusion of the three variables in the final regression model. The characteristics they considered were drainage area, the base flow recession constant and the average basin slope. The final equation, although accurate, cannot be used directly at an ungauged site without additional efforts being required to estimate the base flow constant. Vogel and Kroll suggest that this independent variable could be estimated from maps that would have to be developed or could be obtained through a modest and targeted streamflow gauging program. They suggest that the recession constant could be estimated simply by observing a few recession hydrographs.

Other regional low-flow studies in the United States have used soils characteristics (Carpenter and Hayes, 1996) and the slope of the flow-duration curve (Arihood and Glatfelter, 1991) as explanatory variables in estimating low-flow characteristics. Arihood and Glatfelter (1986) mapped the ratio of the 20-percent to 90-percent flow duration in Indiana for use in estimating low-flow characteristics for ungauged watersheds. Flow-duration curves are discussed in the next section of this paper.

5.8.5 Flow-duration curves

Flow-duration curves of daily discharge show the percentage of days that the flow of a stream is

greater than or equal to given amounts over a given period of record. However, they provide no information on the temporal sequences of the flows at a site or the probability of exceedance or nonexceedance in any given year. Even with this temporal limitation, flow-duration curves have a long history of use in water resources planning and management for a variety of purposes. Some of the most common uses of flow-duration curves are in computing hydroelectric power potential for prime power and secondary power, water-supply and irrigation planning, waste-load allocations and other water-quality management problems. Other uses include the determination of wastewater-treatment-plant capacity, river and reservoir sedimentation studies, instream flow requirements for habitat management and the determination of optimal allocation of water withdrawals from reservoirs. They have also been found to be very simple and useful for graphically illustrating flow characteristics from flood to low flows for a basin. The shape of the curve can vary from basin to basin, reflecting differences in physiography and climatology. They are also useful for illustrating impacts of intervention on water availability and can be used for a host of other purposes.

A flow-duration curve is usually constructed empirically by computing a series of ratios of the number of days in a streamflow record that have discharges greater than or equal to preselected values divided by the total number of days in the record. The ratios, which are estimates of the probabilities, are plotted against their respective discharge values to construct the curve. A duration curve of streamflow will generally plot as roughly a straight line on logarithmic probability paper, such as the one shown in Figure II.5.10. This type of paper gives equal plotting accuracy at all discharges so that differences in low-flow characteristics can be discerned more precisely. Flow-duration curves are sometimes based on weekly or me. Such curves are usually less useful than a daily duration curve.

If the streamflow data are stationary, the derived flow-duration curve should provide the long-term exceedance probabilities for the entire range of flows, which is a useful planning tool. The tails of the flow-duration curve have been found to be sensitive to the number of years used to estimate the curve, which is a form of sampling error. Additional details on construction of flow-duration curves are available in other sources (see, for example, Searcy (1959), Institute of Hydrology (1980), and Vogel and Fennessey (1994).

Flow-duration curves can also be computed for each year, with the average or median of the annual-based flow-duration curves representing the typical curve (Vogel and Fennessey, 1994). These allow the development of confidence intervals and return periods to be associated with the flow-duration curve, and the resultant median annual flow-duration curve is less sensitive to extreme periods of observations that may arise over the history of a site.

The overall shape and, in particular, the shape of the lower portion of flow-duration curve is an indicator of the physiographic, geological and climatic conditions of the basin. Of most interest in low-flow studies is the shape of the lower portion of the flow-duration curve. A low-sloping lower portion implies that the basin is permeable and that the response of the basin to rainfall is not flashy. In contrast, a higher-sloping lower curve implies that the basin is less permeable and probably provides a flashy response for a given input of rainfall. A basin with a higher permeability would also tend to have a higher base flow index than the basin with lower permeability (Zaidman and others, 2003).

Regional relationships can be developed to provide estimates of flow duration for ungauged sites within a homogeneous region (Institute of Hydrology, 1980; Fennessey and Vogel, 1990; Ries, 1994). Multiple regression models similar to those outlined for the estimation of low-flow statistics, such as the Q7,10, can also be developed for this purpose. The dependent variable would be, for example, the

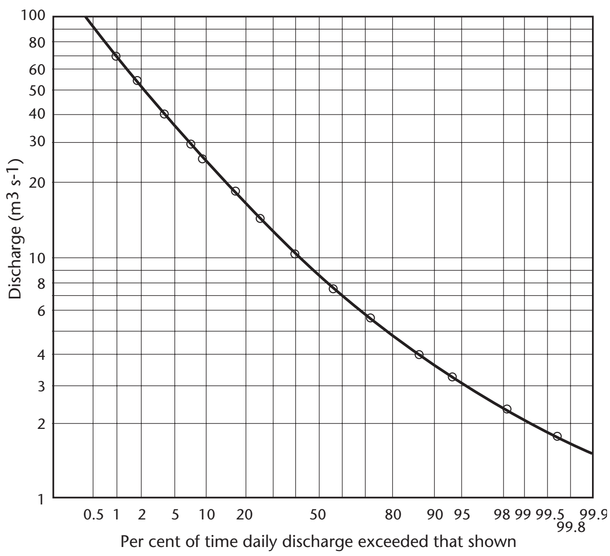


Figure II.5.10. Flow-duration curve of daily discharge

value of the flow exceeded 95 per cent of the time, denoted as Q95 (Institute of Hydrology, 1980). The independent variables of such relationships are also similar to those for other low-flow statistics and would reflect basin characteristics and climatic conditions, such as the drainage area and long-term mean annual precipitation in the basin. HOMS component K10.2.05, regionalization of flow-duration curves, or REGFLOW, can be used to estimate flow-duration curves. It can also be used to relate these to geomorphological characteristics so that flow-duration curves may be estimated for ungauged basins.

5.9 FREQUENCY ANALYSIS OF FLOOD FLOWS [HOMS H83, I81, K10, K15]

In a number of cases, for example, in storage-reservoir design, it is necessary to establish the frequency of flood volumes as well as peak flows. A multivariate statistical analysis of flood hydrographs may be used in this case. A flood hydrograph may be defined by means of the following random variables:

Qmax is the maximum discharge during the flood period; V is the volume (in m^3) of the flood wave; and T is the duration of the flood period.

By using another system of variables, a flood hydrograph may be defined by means of the sequences of discharges $Q_1, Q_2, Q_3, \ldots, Q_n$ corresponding to successive equal intervals of time during the flood period. Statistical analysis of the random variables (Q, V, T) or $(Q_1, \ldots, Q_n)$ may be performed by means of a multivariate probability distribution. Some definitions and computational techniques connected with such probabilistic models may be found in Cavadias (1990). In the case of flood characteristics, a power transformation or other methods may be used to normalize the data. Alternatively, the frequency or probability of occurrence or non-occurrence of a flood volume for an n-day period can be directly estimated by performing a frequency analysis of the site flow data or employing regionalization methods.

The purpose of computing flood and rainfall frequencies is to relate the magnitude of a flood or rainfall depth to its frequency or probability of future occurrence. The key assumptions used to allow interpretation of the frequencies as probabilities are temporal independence of the elements of the analysed sample and stationarity of the record. For flood studies, the use of partial duration series is more questionable than for rainfall, as the different peak floods during the year may be less independent than the corresponding precipitation. However, if care is taken in the selection of the values exceeding a given threshold, a partial duration series analysis should be suitable. The application of frequency analysis to a series of the annual flood maxima – maximum annual series – is more common.

The maximum annual series may be comprised of either daily maxima or instantaneous flood peaks. It is important to distinguish which of the two is required for the analysis. The relation of the two series at a site is dependent on the physical characteristics of the watershed as well as the climatologic factors causing the maxima of both events. For very small drainage areas, it is common that the two maxima do not occur on the same date nor as a result of the same climatic processes acting on the watershed.

Thus, the simplest and most straightforward approach to estimate the frequency of large floods is to use the record available at a site to fit one of the frequency distributions described in 5.1, employing an estimation procedure (see 5.5). Unfortunately, records are not always available at the sites of interest and records may be too short to provide reliable estimates of the rare floods of concern. Thus, most of the discussion in this section addresses the use of information at more than one site to estimate flood quantiles at sites which do not have flood record.

Caution must also be observed in computing frequencies of floods: a clear distinction should be made between stages and discharges. Natural changes in the stage–discharge relationship with time or direct intervention in the channel may render many stage data non-homogeneous and unsuitable for frequency analysis. For most studies, it is preferable to work with discharges, and, if necessary, to then convert the results to stage frequency using an appropriate stage–discharge relationship. In certain cases, such as high stages caused by ice jams, it may be more suitable to work solely with stages for defining flood plains because the flow rate is not an issue.

5.9.1 Regionalization of flood flows

For a site that does not have a large number of observations in its maximum annual series, regional flood frequency analysis is recommended for the estimation of the flood quantiles. Even

with 50 years of data it can be very difficult to regionalize the shape parameter of a distribution. As the record gets shorter, regionalizing the coefficient of variance should be considered. However, the point at which it becomes appropriate to regionalize depends on the homogeneity of the regions that can be constructed and the relative accuracy of at-site estimators, which depends upon the at-site coefficient of variation and the skewness of the flood distribution in the region. Two popular regionalization procedures are the index flood method and the regression-based procedures; Fill and Stedinger (1998) explore the combination of the two. Regional procedures rely on data available from other stations in the same hydrologic region to obtain estimates of flood characteristics at the site of interest. Cunnane (1988) indicated that a regional approach can produce more accurate flood estimates, even when a large number of observations are available at that site. In general, there are two steps in a regional flood frequency procedure:

(a) The delineation of hydrologically homogeneous regions consisting of identification of stations with similar behaviour;
(b) Regional estimation, which involves information transfer from gauged sites to the site of interest within the same region.

Homogeneous regions in can be defined in three different ways, as illustrated by Figure II.5.11:

(a) As fixed geographically contiguous regions;
(b) As fixed geographically non-contiguous regions;
(c) As neighbourhoods, where each target station is associated with its own region.

Regional flood estimation procedures can be defined by considering various combination techniques for the determination of homogeneous regions and a number of regional estimation methods (Stedinger and Tasker, 1986; Burn, 1990; Fill and Stedinger, 1998; Pandey and Nguyen, 1999). GREHYS (1996*a*, 1996*b*) presented the results of an inter-comparison of various regional flood estimation procedures obtained by coupling four homogeneous region delineation methods and seven regional estimation methods. GREHYS (1996b) concluded that the neighborhood approach for the delineation of groups of hydrologically homogeneous basins is superior to the fixed-region approach. Hydrological neighborhoods can be determined by using the region-of-influence method (Burn, 1990) or canonical correlation analysis (Cavadias, 1990; Ouarda and others, 1998). Regional flood estimation can be carried out using the index flood method or multiple regressions.

5.9.2 **Homogeneous region delineation**

5.9.2.1 Region-of-influence method

The region-of-influence method (Burn, 1990), considers each site as the centre of its own region. The identification of a region of influence for a target site is based on a Euclidian distance measure in a multidimensional attribute space. The set of attributes can be related to extreme flow

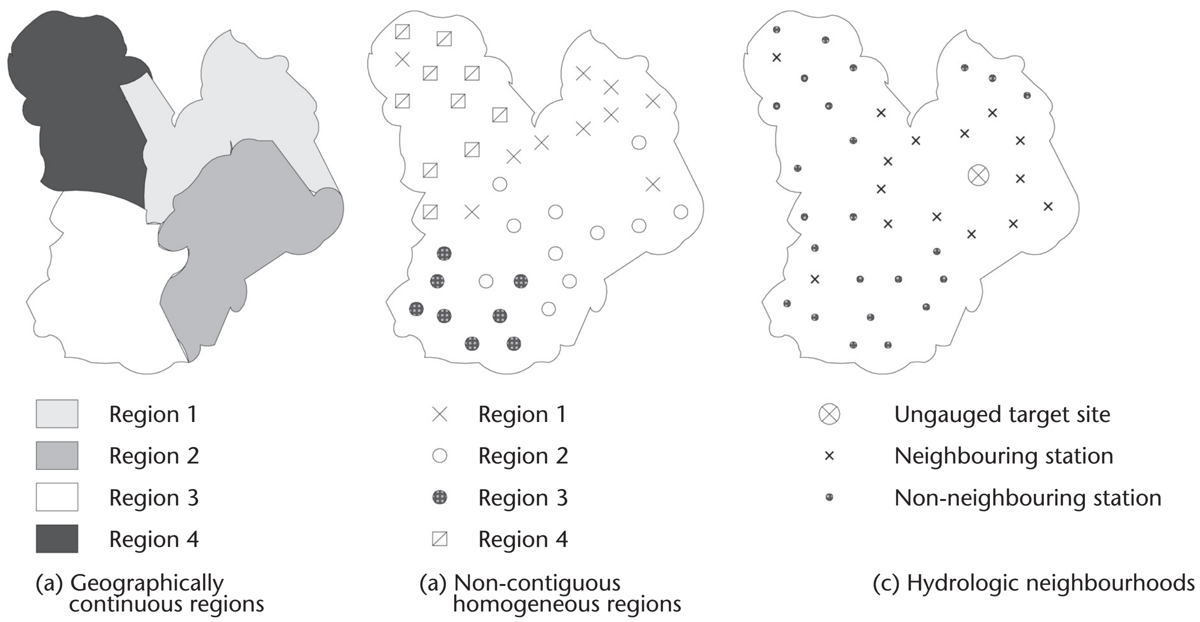


Figure II.5.11. Approaches for the delineation of homogeneous regions (Ouarda and others, 2001)

characteristics of catchments. A weight function is defined to reflect the relative importance of each site for regional estimation at the target site. In the original approach, flow attributes are used to define the region of influence, implying that the site of interest must be gauged. For ungauged sites, climatological and physiographical information may be used as a surrogate for hydrological information. Hence, several versions of the region of influence approach can be considered here, depending on whether the target site is gauged or ungauged, and depending on the space of the attributes. Hydrological attributes that can be considered are the coefficient of variation of maximum floods and the ratio of the mean maximum flow to the drainage area. Other attributes include the longitude, the latitude and meteorological attributes associated with flood events such as the mean total annual precipitation, or the mean snow depth on the ground five days before the spring flood.

The weighted Euclidian distance in the attribute space, D_{ij}, between two sites i and j is given by the following equation:

$$D_{ij} = \sqrt{\sum_{m=1}^{M} \omega_m (C_m^i - C_m^j)^2} \tag{5.51}$$

where M is the number of attributes considered, and $C_m{}^i$ and $C_m{}^j$ are the standardized values of the attribute m for sites i and j. The attributes are standardized by division by their standard deviation over the entire set of stations. The next step is to select a threshold value, ω on D_{ij}, to define the limit of inclusion of stations in the region of influence of a target site.

5.9.2.2 Canonical correlation analysis

Canonical correlation analysis is a multivariate statistical technique that allows a reduction in the dimensionality of linear dependence problems between two groups of variables. This method can be used to identify sites with flood regimes similar to the target site (Cavadias, 1990; Ouarda and others, 1997).

Ouarda and others (1997) demonstrated that the multiple regression method and the index flood method give equivalent results when combined with the canonical correlation analysis. Ouarda and others (1999) presented an automated and transposable regional procedure based on canonical correlation analysis and multiple regressions. The general methodology presented in Ouarda and others (2000) allows the joint regional estimation of flood peaks and flood volumes. A more detailed description of the canonical correlation analysis methodology for regional frequency estimation is available in Ouarda and others (2001). A general description of the method can be found in Muirhead (1982).

5.9.3 Regional flood estimation methods

The second step of regional analysis consists in inferring flood information, such as quantiles, at the target site using data from the stations identified in the first step of the analysis. Regional estimation can be accomplished using the index-flood or regression methods.

5.9.3.1 The index-flood procedure

The index-flood procedure consists of two major steps. The first is the development of the dimensionless frequency curve for a homogeneous region. The curve is derived from individual frequency analyses of all sites. The curve for each site is made dimensionless by dividing the curve by an index, such as the flood corresponding to the two-year or 2.33-year return period or the mean. The median dimensionless values are selected for the sites for various return periods. They are in turn plotted on probability paper. The second step consists of the development of a relationship between the index and the physical and climatological characteristics of the watershed using regression-based procedures. The combination of the index with the dimensionless curve provides a frequency curve for any watershed within the region.

Much work has been done to extend these initial concepts and assess the accuracy of index procedures to determine various flood quantiles, for example in Gabriele and Arnell (1991). Advances have been facilitated by the development of probability-weighted-moment (Greanwood and others, 1979) and L–moment (Hosking, 1990) statistics. The need for analytical homogeneity tests can be circumvented by the use of Monte Carlo experiments. Homogeneity should and can be extended from the slope of the curve, which is the coefficient of variation of the sample in Dalrymple's approach, to include the skewness and kurtosis of the proposed region. This leads to a more flexible index procedure that allows higher moments of the region's data to indicate the potential underlying distribution. Heterogeneity of the lower moments can be assessed and potentially linked to characteristics of the watershed. Hosking and Wallis (1988) show that "even when both heterogeneity and intersite

dependence are present and the form of the [regional] flood-frequency distribution is mis-specified, regional flood frequency analysis is preferable to at-site analysis". The index-flood method has been found to be one of the most efficient regionalization techniques.

5.9.3.2 **Regression-based procedures**

Regression techniques can be used to estimate the magnitude of a flood event that will occur on average once in *Tr* years, denoted Q_{TR}, by using physical and climatological watershed characteristics. The magnitudes of flood events for various return periods for each gauging station are estimated by using a preselected distribution from an at-site frequency analysis. In turn, characteristics for each watershed are derived from topographic maps or from generalized climatological data. The parameters of the equations that relate Q_{TR} to the characteristics can be obtained by using ordinary least squares, weighted least squares or generalized least squares techniques. The latter two approaches have been used to overcome the deficiencies in the assumptions of ordinary least squares. Ordinary least squares regression procedures do not account for variable errors in flood characteristics caused by unequal record lengths at gauging stations. Tasker (1980) proposed the use of weighted least squares regression with the variance of the errors of the observed flood characteristics estimated as an inverse function of the record length. Generalized least squares have been proposed because they can account for both the unequal reliability and the correlation of flood characteristics that exist between sites. Using Monte Carlo simulation, Stedinger and Tasker (1985 and 1986) demonstrated that the generalized least squares procedure provides more accurate estimates of regression coefficients, better estimates of the accuracy of the regression coefficients and better estimates of the model error.

The regional flood–frequency relationship developed by Benson (1962) for the north-eastern United States is as follows:

$$Q_{TR} = aA^b Z^c S^d P^e D^f M^g \tag{5.52}$$

where Q_{TR} is the T-year annual peak discharge, A is the drainage area, Z is the main channel slope, S is the percent of surface storage area plus 0.5 per cent, P is the T-year rainfall intensity for a particular duration, D is the average January degrees below freezing, M is an orographic factor, and a, b, c, d, e, f and g are regression coefficients. Many independent variables were tested to derive equation 5.52 and many definitions were tested for each variable. The goal is to obtain independent variables that are physically related to the dependent variable. Independent variables that are related to a low return-period flood may not be a driving force behind a higher return-period flood. A logarithmic transformation of equation 5.47 may be taken to create a linear additive model for the regression procedures. Other types of transformations could be applied to the dependent and independent variables, but the logarithmic transformation remains the most popular. Both the signs and the magnitude of the coefficients of the model should make hydrological sense. For example, the exponent d of the surface-storage term should be negative because of the effect of such storage (lakes, reservoirs and so forth) in flattening out flood peaks. Other exponents should be positive with their magnitudes varying with the return period. Care should be taken to ensure that not too many independent variables are included in the model. The variables included in the regression model should be statistically significant at some preselected and generally accepted level of significance (Draper and Smith, 1981).

The resulting regression equation should be evaluated to determine if it is regionally homogeneous. Residual errors of the regression should be plotted on topographic maps to check visually if geographic biases are evident. If a bias in the estimation of the T-year annual peak discharge is geographically evident, then the Wilcoxon signed-rank test can be applied to test its significance. The test provides an objective method for checking the hypothesis that the median of the residuals in a sub-region is equal to the median residual of the parent region for which the regression equation was computed. Different homogeneous regions may be found for different return periods. The homogeneous region for the relationship linking the index flood to the characteristics of the watershed need not coincide with the homogeneous region for the characteristics of the distribution of the index method, such as the slope of the dimensionless curve.

In practice, the power form function is the most commonly used model to describe the relationship between the at-site estimates of flood quantiles Q_T and the hydrometeorological and basin characteristics for the region identified in the first step of the procedure. A common procedure for the estimation of the parameters consists in lineralizing the power relationship by a logarithmic transformation, and then estimating the parameters of the linearizing model by an ordinary least squares technique. The usual procedure is therefore straightforward, because one can make use of multiple linear

regression techniques to identify the parameters of a nonlinear model.

An advantage of the multiple regression regional estimation models is the flexibility in choosing the type of distribution to represent the exceedances at each site. The regression-based regional estimation method can also be applied using peaks-over-threshold data, in which case the generalized Pareto, exponential, and Weibull distributions can be used. Both the generalized Pareto distribution and the Weibull distribution contain the less flexible exponential distribution as a special case. In the peaks-over-threshold approach, all flood peaks above a prefixed threshold are considered. The lack of detailed guidelines for choosing the most appropriate threshold constitutes a serious drawback of the method and is probably one reason why it is less used in practice than its annual flood series counterpart. For a review of various methods for threshold selection, see Lang and others (1999).

A regression-based method can also be performed using non-parametric frequency analyis, which does not require a priori distribution selection. Adamowski (1989) and Guo (1991) found that non-parametric methods are particularly suitable for multimodal annual flood data following mixed distributions. Non-parametric density estimation has been used successfully in a regional framework (GREHYS, 1996*b*), including non-parametric regression (Gingras and others, 1995). As well, the L–moments technique can be used at all stages of regional analysis including homogeneous region delineation and testing, identification and testing of regional distributions and quantile estimation (Hosking and Wallis, 1997).

5.9.4 At-site and regional flow–duration–frequency approach

Most of the regional flood frequency analysis literature describes a flood event only by its instantaneous peak or its maximum daily flow. When designing a hydraulic structure or mapping a flood plain, information about flood peaks is essential, but more information may be desired. Indeed, flood severity is not only defined by the flood's peak value, but also by its volume and duration. The analysis of flow duration frequency, or QDF (Sherwood, 1994; Javelle, 2001), also known as flood duration frequency or discharge deviation frequency, has been proposed as an approach for a more thorough description of a flood event. Flow–duration–frequency analysis is similar to the intensity–duration–frequency analysis commonly utilized for rainfall (see 5.7 above). In this case, averaged discharges are computed over different fixed durations D. For each duration, a frequency distribution of maximum discharges is then analysed. Finally, a continuous formulation is fitted as a function of the return period (T) and the duration (D) over which discharges have been averaged. Javelle and others (2002) proposed a converging flow–duration–frequency model based on the assumption of convergence between the different discharge distributions for small return periods. This formulation has been successfully tested for basins located in France, Martinique and Canada.

Javelle and others (2002) have also presented a regional flow–duration–frequency approach, combining the local flow–duration–frequency formulation presented by Javelle (2001) and the index flood method outlined in 5.9.3.1, which is commonly used in regional flood frequency analysis. This regional model was developed by Javelle and others (2003) for 169 catchments in the Canadian provinces of Quebec and Ontario, and it was used to define different types of flood behaviour and identify the corresponding geographic regions. Javelle and others (2003) showed that the parameters of the regional flow–duration–frequency model provide information about the flood dynamics. Unlike the intensity–duration–frequency analysis for rainfall, flow–duration–frequency analysis remains under-utilized despite its strong potential.

5.9.5 Combination of single-site and regional data

The objective of procedures that aim to combine single-site and regional information is to improve upon at-site estimates that are based on a limited series of site data by using available information from other sites. The need for such procedures is particularly great in the estimation of extreme hydrological phenomena where a combination of limited site data and inference in the tails of probability distributions conspire to destabilize such estimators. A simple Bayesian approach presented by Fortin and others (1998) combines local and regional quantile estimates knowing the variance of estimation for each estimate. The United States has guidelines for combining at-site quantile estimates obtained by regional regression using the standard error of each (Interagency Advisory Committee on Water Data, 1982). The approach presented by Kuczera (1982) and evaluated by Lettenmaier and Potter (1985) is based on an empirical Bayes model that combines an at-site and regional variance and was shown to lead to substantial improvements in performance over procedures that only used at-site information.

Clearly, regional hydrological information should be of value in improving flood estimates, particularly with regard to the shape and characteristics of the tail of the distribution, as these are hard to resolve with limited at-site datasets. For this reason, procedures adopted in many countries employ some combination of at-site skew, as well as the at-site mean and standard deviation so as to estimate a flood frequency distribution. In certain cases, only the skew is regionalized, and the regional skew is average with at-site skew. In the United Kingdom, the general procedure is to use an index flood procedure that uses the at-site mean with a regional growth curve to define flood risk at a gauged site (Robson and Reed, 1999), so that the value of two parameters of the fitted logistic distribution are determined by regional data.

Striking the right balance between the use of regional information and at-site records to define the frequency curve is a challenge. Clearly the less data one has at a site, the less confidence one has in at-site estimation of statistics, and the more the weight that should be placed on regional information.The at-site standard deviation can also be weighted with a regional value (Kuczera, 1982; Lettenmaier and Potter, 1985) or the at-site mean and standard deviation cab be used with a regional shape estimator (Lettenmaier and others, 1987).

Region-of-influence ideas are appropriate here in defining the set of sites used for pooling. Using regional estimators of the coefficient of variation and skewness based on different spatial averaging scales in a hierarchical approach (Gabriele and Arnell, 1991) or regression to describe how a growth curve or a shape parameter varies continuously with basin characteristics (Madsen and Rosbjerg, 1997) are available options. The appropriate choice depends upon the homogeneity or heterogeneity of the region and other flood distribution characteristics, the length of the record available at different sites and the time an agency has to determine and understand those trade-offs. Stedinger and Lu (1995) illustrate some of the trade-offs among the number of regionalized parameters, the length of record and the number of sites available, regional heterogeneity and flood distribution characteristics.

5.9.6 Flood frequency analysis and climate variability

The foregoing discussion has for the most part embodied the traditional assumption that flood series are a set of independent and identically distributed random variables. If they are not entirely independent but instead have some modest correlation from year to year, it has relatively little impact on the analysis and the bias of estimated flood quantiles. The more troubling concern is either a trend in the distribution of floods due to development and other changes in the basin, or what has been called climate variability and climate change. All three of these effects can have a significant impact on flood risk in a basin.

The easiest of the three to deal with is when changes in the basin – particularly land cover, the drainage network and channel characteristics – or the construction and operation of detention structures have evolved over time. A traditional record of annual maximum floods is no longer effective in describing the risk of flooding under the new regime. The traditional approach to handling changes in channel characteristics and the operation of storage structures is to route a historical record of natural flows through a hydraulic model to generate a record of regulated flows, which can be used as a basis for frequency analysis. Alternatively, a frequency analysis can be conducted on the natural flows and a design natural flow hydrograph can be constructed that is, in turn, routed through the hydraulic model based on the assumption that owing to operation of the facility, the exceedance probability of the design hydrograph would be unchanged because smaller and larger events would have resulted in smaller and larger flood peaks downstream, respectively.

For complicated systems involving several streams or storage facilities, or for basins that have experienced significant land-cover and land-use change, it is advisable to use historical or synthetic rainfall and temperature series to drive physically based rainfall-runoff and hydraulic models. Such a study allows the analyst to appropriately describe the operation of different facilities, network and channel modifications, as well as the likely effect of land-cover and land-use changes.

Dealing with climate variability and climate change is a difficult problem (Jain and Lall, 2001). NRC (1998) makes the following observation:

> Evidence accumulates that climate has changed, is changing and will continue to do so with or without anthropogenic influences. The long-held, implicit assumption that we live in a relatively stable climate system is thus no longer tenable.

Changes in hydroclimatological variables, both rainfall and runoff, over different timescales are

now well documented for sites around the world (Hirschboeck and others 2000; Pilon and Yue, 2002; Pekarova and others, 2003). Two cases are immediately clear, those corresponding to climate variability and climate change.

The first concern, climate variability, relates to such processes as the El Nino-Southern Oscillation or the North Atlantic Oscillations, which result in a sporadic variation in the risk of flooding over time on the scale of decades. In cases where the record is relatively, it would be hoped that such phenomena would have passed through several phases resulting in a reasonable picture of the long-term average risk. With short records, such variations are more problematic. It is always good practice to attempt to use longer records from the same region in order to add balance to the short record. If a composite or a cross-correlation between the short record and longer records in the region is reasonably high, record augmentation methods described in 5.5.4 can be used to develop a more balanced, long-term description of flood risk. However, with smaller catchments where year-to-year events are highly variable, it may not be effective to use simple record augmentation to correct distinct differences in flood risk between different periods because the cross-correlation between concurrent annual peaks will be too small.

For operational concerns, an option would be to forecast variations in the El Nino-Southern Oscillation, or other indices, and unexplained hydrological variations, so as to forecast more accurately the flood risk in the current and subsequent years and advise water operations accordingly (Piechota and Dracup, 1999). However, for project planning purposes such short-term variations are likely to be too short lived to affect the economic design of projects.

The second climate concern would be climate change in one direction or another that is not quickly reversed within a decade or two. Such climate change is on the scale of decades and is a very serious concern. Even mild upward trends can result in substantial increases in the frequency of flooding above a specified threshold, as shown by Porparto and Ridolfi (1998) and Olsen and others (1999). It is clear that anthropogenic impacts are now inevitable. The question is how soon and how severe. Guidance is much more difficult to provide for this case because there is no clear consensus on how fast the Earth is likely to warm from the release of greenhouse gases into the Earth's atmosphere and what the impact of those changes will be on meteorological processes at a regional or watershed scale. Generalized circulation models of the Earth's atmosphere have given some vision of how local climates may change, but the inability of such models to capture current meteorological processes at a regional or watershed scale yields limited confidence that they will be able to predict accurately the rate and intensity of future change. However, the hydrological implications of different generalized circulation model scenarios are often investigated to provide a vision of what the future may hold (see Arnell and others (2001)). And, as Arnell (2003) points out, the future will be the result of both natural climate variability and climate change.

5.10 ESTIMATING DESIGN FLOODS [HOMS K10, K15, I81, K22]

5.10.1 General

The 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; its selection involves choosing safety criteria and estimating the flood magnitude that meets the criteria. The risk of damage occurring is equivalent to the probability of occurrence of floods larger than the design flood. The decisive factor in the determination of a design flood is that feature or parameter of the flood that can be identified as the major cause of potential damage. The decision as to which is the most relevant flood parameter for a particular project lies with the planner and the designer and should be based on an engineering analysis of the given situation. Decisive parameters usually include the following:

(a) Peak discharge in the case of culverts, storm sewers, bridge openings, spillways and outlets of weirs and small dams;
(b) Peak stage in the case of levees, clearance under bridges, flood-plain zoning and the design of roads and railways in river valleys;
(c) Flood volume for the design of flood-control reservoirs and, generally, for all cases where flood attenuation by water storage can be significant, such as for the design of spillway capacities and freeboards on dams;
(d) Flood hydrograph shape in cases where superposition of several floods must be considered, such as for flood protection downstream from the mouth of large tributaries or for reservoir operation during floods.

5.10.2 Design floods

The following types of design flood are commonly used in water-resource engineering practice (Singh, 1992):

(a) Spillway design flood – a term often used in dam design to identify a flood that a spillway must be able to pass to provide the desired degree of protection for a dam;
(b) Construction flood – a flood for which reasonable precautions will be taken to avoid flooding of construction sites and thereby to prevent damage to a project during its construction;
(c) Probable maximum flood – the largest flood that may be expected at a site, taking into account all pertinent factors of location, meteorology, hydrology and terrain (see 5.7). It essentially has an infinite return period and can be selected as the design flood to prevent a major disaster;
(d) Standard project flood – a flood resulting from the most severe combination of meteorological and hydrological conditions that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations. It has a long but unspecified return period and may be selected as a design flood for structures of great importance;
(e) Frequency-based flood – a flood determined employing frequency analysis of flood flows or rainfall data by performing one of the following:
    (i) frequency analysis of rainfall data to estimate a frequency-based design storm, which is then converted to design flood;
    (ii) frequency analysis of flood flows available at the site to directly estimate the design flood;
    (iii) regional frequency analysis to estimate the design flood.

5.10.2.1 Magnitude and methods of computation

A design flood can be estimated by transforming the design storm to design flood using, for example, the unit hydrograph concept or flood frequency analysis. The latter requires long-term streamflow data at the site of interest. If streamflow data are unavailable or a hydrograph is required, then the design flood can be estimated using either a rainfall frequency analysis coupled with a rainfall-runoff model or a rainfall-runoff method that may be either data-based, or hypothetical or empirical. The rainfall information used for design flood estimation is referred to as the design storm and can be classified as probable maximum precipitation, a standard project storm, or a frequency-based storm. For structures involving low-damage risk, such as culverts and secondary roads, the design flood may be calculated by empirical methods, given the typically low return period of such structures and their relatively low importance. For structures or projects involving major potential damage, but without a risk of loss of life, design floods should be computed, if possible, by methods allowing an evaluation of their return periods so that optimization methods can be used for the selection of the design flood magnitude. For situations involving a risk of loss of life, the aim is to provide maximum protection, and the maximum probable flood or the standard project flood is usually adopted as the design flood. It is advisable to evaluate the reasonableness of the probable maximum flood by comparing it with observed rainfalls and floods.

Only a few of the more practical and popular methods for calculating floods have been described in this chapter. There are many other methods, some of which have been developed for particular regions, such as those described by Maidment (1993) and Kundziewicz and others (1993). For example, the GRADEX method (Guillot, 1993; Ozga-Zielinski, 2002) is based on the combined use of rainfall and flow records. It assumes that the upper tail of the flood is near an exponential asymptote (gradient) of rainfall. The *Flood Estimation Handbook* proposes a procedure developed by the Centre for Ecology and Hydrology in the United Kingdom that combines statistical analysis and modelling of precipitation time series to the hydrological simulation of discharge at catchment scale (www.nerc-wallingford.ac.uk).

5.10.2.2 Design life of a project and design criteria

In the wide range of cases in which the design flood is selected by optimizing the relation between the expected flood damage and the cost of flood-protection measures, the resulting optimum level of the calculated risk depends to a certain degree on the length of the period over which the performance of the project is evaluated. This period is called the design life or planning horizon of the project and is determined in the project-planning stage on the basis of the following four time spans:

(a) Physical life, which ends when a facility can no longer physically perform its intended function;
(b) Economic life, which ends when the incremental benefits from continued use no longer exceed the incremental cost of continued operation;

(c) The period of analysis, which is the length of time over which a facility may be expected to function under conditions that can be relatively accurately foreseen at the time of the analysis; any operation in the distant future that is subject to a high degree of uncertainty is excluded from consideration;
(d) The construction horizon, which is reached when a facility is no longer expected to satisfy future demands, becoming functionally obsolete.

The optimum level of calculated risk, hence the design return period for each of these periods may be different. The final selection of the design flood cannot be made without considering political, social, environmental and other quantifiable criteria.

In many cases, flood analysis criteria are often prescribed by regulations and not subject to negotiation. Different types of projects may require different types of criteria reflecting economic efficiency and safety. Safety criteria can be specified in terms of a return period, meteorological input and/or the maximum flood on record. The return period (T), in years, that is to be used is often specified by the competent agency and may be related to specified risk (R) or probability of failure (per cent) over the service life (n) (in years) as given by $T = 1/[1 - (1–R)^{1/n}]$ (see 5.10.8).

For example, when n = 2 and the acceptable risk is $R = 0.02$ per cent, then $T = 99.5$ years. A distinction should be made between specifying the criteria that is to be met and specifying the computational method to be used to estimate the design flood. When the computational method is not specified by the regulation, it must be selected and justified by the designer. It is advisable to ensure the adequacy of the design against given conditions and intent of the project.

5.10.2.3 Design floods for large reservoirs

The selection of design floods for the spillway design of large storage reservoirs must be given special attention because a reservoir may considerably change the flood regime, both at the reservoir site and in the downstream section of the river.

The basic flood-related effect of a reservoir is flood attenuation. Its estimation requires knowledge of the original flood hydrograph shape. When the hydrograph is not known, a hypothetical shape, often triangular, is assumed and fitted to the selected flood volume and peak discharge. In evaluating the effect of flood attenuation on the reduction of spillway capacity and freeboard of a dam, it is imperative to adopt a conservative approach and to consider only those effects that can be guaranteed at all times. Thus, only the effect of the ungated spillway should be considered. All gated outlets should be assumed to be closed and the reservoir filled to the crest of the fixed spillway at the beginning of the flood.

In addition to flood attenuation, the flood regime downstream must be analysed carefully from the point of view of changes in the timing of flood peaks, the effect of changes in the shape of flood hydrographs and the effects on the river channel caused by an increased scouring tendency of the virtually sediment-free water leaving the reservoir through the spillway.

The type of dam structure must also be considered because it is of prime importance in determining the vulnerability of the dam should overtopping occur. Vulnerability is highest for earthfill dams, which are in great danger of collapsing if overtopped.

5.10.2.4 Probable maximum flood

Probable maximum flood is computed from probable maximum precipitation (see 5.7) or from the most critical combination of maximum snowmelt (see 6.3.4) and rainfall, and it provides an indication of the maximum possible flood that could reasonably be expected for a given watershed. It is not possible to quantify the term reasonable or assign a long but arbitrary return period to the probable maximum flood. The concepts of probable maximum precipitation and probable maximum flood are controversial. Nevertheless, it is necessary to assess the potential impact of such extreme events; therefore, numerical flood estimates are required for very extreme floods and are often used in design practice.

Probable maximum precipitation is analytically estimated as being the greatest depth of precipitation for a given duration that is physically plausible over a given watershed at a certain time of the year, and its estimation involves the temporal distribution of rainfall. The concepts and related methodologies are described by WMO (1986*a*). The US Army Corps of Engineers (1985) has a computer program, HMRS2, to compute probable maximum precipitation, which can then be used with HEC-1 (see 5.10.5) to determine probable maximum flood. WMO (1969) provides more

details on estimation of maximum floods (see 6.3.2).

As rainfall usually accounts for a major portion of probable maximum flood runoff, special consideration must be given to the conversion of rainfall to runoff. This conversion is done by deterministic rainfall-runoff models, but their application for this purpose involves certain modifications designed to accommodate the extreme magnitude of the rainfall event that is being used as input. The most important modifications are as follows:

(a) The effect of the initial soil-moisture conditions and of the variation of the infiltration rate during the rainfall on streamflow is greatly reduced, compared to their effect in streamflow simulation under normal conditions. Hence, the refined methods employed in most models for estimating infiltration indices can be considerably simplified. A common practice is to use the minimum infiltration capacity, or the maximum runoff coefficient, for a given soil type and vegetation cover, throughout the entire storm;

(b) When a unit hydrograph is used to transform the maximum precipitation, it should be remembered that the validity of the underlying assumption of linearity is limited to conditions similar to those for which the unit hydrograph was derived. An analysis of floods in a number of basins (Singh, 1992) has shown that the peak ordinates of unit hydrographs derived from major floods (greater than 125 mm of runoff over the basin area) are often 25 to 50 per cent higher than peak ordinates derived from minor floods (25 to 50 mm of runoff). It is important to bear in mind that the adjustment of the unit hydrograph for the computation of the probable maximum flood must be guided by the necessity of making a conservative estimate: one that leads to the greater flood;

(c) In the case of drainage basins larger than 500 km^2, or even smaller basins where their different parts have widely different runoff characteristics, it is generally necessary to derive separate unit hydrographs and probable maximum floods for several sub-areas and to obtain the probable maximum flood for the whole basin by routing the component floods downstream to the project site. It must be remembered that the same positioning of the isohyetal pattern of the design storm over the catchment, which yields the maximum flood if a single unit hydrograph is used for the whole catchment, need not yield the maximum flood if the catchment is subdivided into several sub-areas. Thus, for each different catchment subdivision, an optimal positioning of the design storm, that is, the position yielding the most unfavourable combination of the relevant parameters of the probable maximum flood, must be found separately subject to the restrictions due to orography, as discussed in 5.7. The optimal position of the design storm can be obtained as a result of sensitivity analysis.

Although no specific return period can be assigned to the probable maximum flood, its parameters should be compared with the respective frequency curves fitted to historical floods to make sure that they have extremely long return periods and have been unequalled by any historical flood event.

5.10.2.5 Standard project flood

A standard project flood is usually about 50 per cent of a probable maximum flood (Singh, 1992). Its determination is governed by considerations similar to those relevant to the probable maximum flood. The standard project flood is usually determined by the transformation of the transposed largest rainstorm observed in the region surrounding the project, rather than from a meteorologically maximized rainstorm, as in the case with the probable maximum flood. Nonetheless, the standard project flood should represent a very rare event and should not be exceeded by more than a few per cent by the major floods experienced within the general region.

5.10.3 Data preparation

Basic data for determining design floods are the records collected by regional or national Hydrological and Meteorological Services. These data exist in the form of stage recordings and discharge measurements that form the basis for the computation of rating curves. As the magnitude of the design flood depends primarily on measurements of high discharges, special attention should be given to their evaluation and the extension of rating curves.

For a proper assessment of the flood regime, it is essential to obtain sufficient information on historic floods. The basic element of such information is stage. In compiling information on flood stages, use can be made of traces of materials deposited by floods, flood marks on bridges, buildings and river banks; recollection of long-time residents; photographs taken during floods; archived materials; articles in the press and memoirs. Paleoflood

information can also be considered (Viessman and Lewis, 2003).

To convert flood stages determined by such investigations into discharges, hydraulic computations must be based on reconstructed river cross-sections, longitudinal profiles, the slope of water surface and channel roughness. All the known modifications of the river channel should be taken into account, such as dredging, embankments and channel straightening. Owing to the limited accuracy of the reconstructed river characteristics, the application of the Manning and Chézy formulae is generally satisfactory for hydraulic computations of this kind. Software such as HEC-RAS can facilitate the analysis.

5.10.4 Design flood computation techniques

The selection of computational techniques for the determination of design floods depends on the type, quantity, and quality of available hydrological data, as well as the type of design flood information. Owing to the complexity of the flood producing process, the estimates are only approximations, and understanding of related issues is important to produce reliable estimates. There are many methods, and the choice is often made on a subjective and intuitive basis. Some practical criteria for the choice of the method can be found in Pilgrim and Doran (1993) and details of many methods are available in Pilgrim and Cordery (1993), Bedient and Huber (2002) and Viessman and Lewis (2003).

Depending on data availability and design requirements, the methods of estimating design floods can be grouped into empirical, frequency-based and rainfall-runoff methods.

To extract maximum information from scarce or inaccurate data, it is advisable to apply several different methods, compare the results and choose the design parameters based on engineering judgment. Sensitivity analysis can be useful in making the final decision because it may show the impact of potential errors on the magnitude of the design variable.

5.10.4.1 Empirical methods

Empirical flood formulae expressed as a flood envelope curve may be used to provide a rough estimate of the upper limit of discharge for a given site. A common type of formula expresses the peak discharge Q ($m^3\ s^{-1}$) as a power function of catchment's area A (km^2) (Bedient and Huber, 2002),

$$Q = CA^n \tag{5.53}$$

where coefficient C and exponent n vary within wide limits and the values for a particular study can be selected on the basis of empirical data.

The application of empirical formulae is generally limited to the region for which they have been developed, and they should be used with great caution and only when a more accurate method cannot be applied. Another drawback of empirical formulae is the difficulty in assessing the return period of the computed peak flow.

An envelope curve enclosing maximum observed peak flows can be plotted against catchment areas for a large number of stations in a meteorologically and geomorphologically homogeneous region. Such curves provide useful information, especially in cases where few data are available at any single station. Attempts have been made to refine the technique by constructing various envelopes related to different climatological and/or geomorphologic factors. However, the return periods of the peak flows remain unspecified. Uses of such formulae provide a rough estimate providing only the order of magnitude of large flood flows.

5.10.4.2 Rainfall-runoff models

Depending on whether the design flood is to be synthesized from precipitation and/or snowmelt or from known flood hydrographs at upstream points, the models of interest fall into two broad categories:

(a) Rainfall-runoff models, as described in 6.3.2;
(b) Streamflow routing models, as described in 6.3.5.

Many rainfall-runoff relationships have been developed that could apply to any region or watershed under any set of conditions. However, these methods should be used with caution, as they are only approximate and empirical. The most widely used practical methods are the unit hydrograph method (see 6.3.2.3), the rational method (see below), the Soil Conservation Service (SCS) method (see below) and conceptual models (see 5.10.5).

5.10.4.2.1 *Rational method*

One of the oldest and simplest rainfall-runoff formulae is the rational formula, which allows for

the prediction of peak flow Q_p ($m^3\ s^{-1}$) from the following equation:

$$Q_p = 0.278CiA \tag{5.54}$$

where C is the runoff coefficient that is dimensionless and selected according to the type of land use in the watershed, i is rainfall intensity (mm/hr) of chosen frequency and for duration equal to the time of concentration, and A is the watershed area (km^2). This method is often used in small urban areas as well as for rough estimates in rural areas in the absence of data for other methods. It is highly sensitive to rainfall assumptions and the selection of C. Use of this method should be restricted to small areas; although the upper limit is not explicitly established, it varies between 40 ha and 500 ha.

Because of its predominant use in urban areas, the rational method is dealt with in more detail in 4.7.

5.10.4.2.2 *Soil Conservation Service method*

The former US Department of Agriculture Soil Conservation Service, now the National Resource Conservation Service, suggested an empirical model for rainfall abstractions based on the potential for the soil to absorb a certain amount of moisture. On the basis of field observations, the potential storage S was related to a curve number CN varying between 0 and 100, which is a characteristic of the soil type, land use and the initial degree of saturation known as the antecedent moisture condition (AMC). The value of S is defined by the empirical expression:

$$S = 25.4\left(\frac{1000}{CN} - 10\right)\text{(millimetres)} \tag{5.55}$$

The values of CN are given in Table II.5.8 as a function of soil type (A, B, C, D), land use, hydrological condition of the watershed and antecedent moisture condition (AMC I, II, III).

According to this method, the depth of surface runoff is given by the following equation:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{5.56}$$

where Q is the depth of surface runoff, P is the accumulated depth of rainfall, I_a is an initial abstraction: no runoff occurs until accumulated rainfall exceeds I_a, and S is the potential storage in the soil.

All units are in mm, and for values of $P > I_a$. Using observed data, the Natural Resources Conservation Service found that I_a is related to S, and on average is assumed to be $I_a = 0.2S$; thus the equation becomes:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{5.57}$$

for $P > 0.2S$, and $Q = 0$ when $P \le 0.2S$. Since initial abstraction consists of interception, depression storage and infiltration prior to the onset of direct runoff, the value of I_a can be modified to account for local conditions.

Soils are classified as A, B, C, and D according to the following criteria:

(a) Group A soils have low runoff potential and high infiltration rates, greater than 7.6 mm/hr, and consist primarily of deep well-drained sands and gravel;

(b) Group B soils have moderate infiltration rates (3.8–7.6 mm/hr) and consist primarily of moderately fine to moderately coarse textured soils, such as loess and sandy loam;

(c) Group C soils have low infiltration rates (1.27–3.8 mm/hr) and consist of clay loam, shallow sandy loam and clays;

(d) Group D soils have high runoff potencial and low infiltration rates (less than 1.27 mm/hr) and consist primarily of clays with high swelling potential, soils with a permamnt high water table or shallow soils over nearly impervious material.

CN values for urban and composite areas should be determined.

The runoff from a particular rainfall event depends on the moisture already in the soil from previous rainfall. The three antacedent moisture conditions are as follows:

(a) AMC I – Soils are dry but not to wilting point;

(b) AMC II – Average conditions;

(c) AMC III – Heavy rainfall or light rainfall with low temperature have occured within the last five days saturating the soil.

Table II.5.8 provides CN(II) values for average condtions AMC II. CN(I) and CN(III) corresponding to AMC(I) and AMC(III) can be estimated from:

$$CN(I) = 4.2CN(II)/(10 - 0.058CN(II) \tag{5.58}$$

and

$$CN(III) = 23CN(II)/(10 + 0.13CN(II) \tag{5.59}$$

Table II.5.8. Runoff curve numbers for selected agricultural, suburban and urban land use (AMCII, and I_a = 0.25) (after Bedient and Huber, 2002)

		Hydrological soil group			
Land-use description		*A*	*B*	*C*	*D*
Cultivated land[a]					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or rangeland					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover, no mulch		45	66	77	83
Good cover[b]		25	55	70	77
Open spaces: lawns, parks, golf courses and so forth					
Good condition: grass cover = 75% or more		39	61	74	80
Fair condition: grass cover = 50–75%		49	69	79	84
Commercial and business areas (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential[c]					
Average lot size	Average % impervious[d]				
1/8 acre[e] or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways and so forth[f]		98	98	98	98
Streets and roads					
Paved with curbs and storm sewers[f]		98	98	98	98
Gravel		76	85	89	91
Dirt		72	82	87	89

a For a more detailed description of agricultural land-use curve numbers, please refer to *National Engineering Handbook* (Natural Resources Conservation Service, 1972).
b Good cover is protected from grazing and litter and brush cover soil.
c Curve numbers are computed assuming that the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur.
d The remaining pervious areas (lawns) are considered to be in good condition for these curve numbers.
e 1 ha = 0.404687 acre
f In some warmer climates of the country a curve number of 95 may be used.

5.10.4.2.3 *Soil Conservation Service unit hydrograph*

The earliest Soil Conservation Service method assumed that a hydrograph is a simple triangle, as shown in Figure II.5.12, with rainfall duration D (hours), time to peak TR (hours), time of fall B (hours) and the peak discharge Q_p(m^3 s^{-1}) given by the following equation (Bedient and Huber, 2002):

$$Q_p = \frac{0.208\ AQ_R}{T_R} \tag{5.60}$$

where A is the watershed area (km^2) and Q_R indicates the runoff depth for unit hydrograph calculations (mm). Figure II.5.12 shows that the time to peak (hours) is as follows:

$$T_R = D/2 + t_p \tag{5.61}$$

Where D is the rainfall duration (in hours) and t_p is the lag time (in hours) from centroid of rainfall to Q_p(m^3 s^{-1}). Lag time t_p is estimated from any one of several empirical equations used by the SCS, such as:

$$t_p = \frac{l^{0.8}\,(S+1)^{0.7}}{1900\,y^{0.5}} \tag{5.62}$$

where l is the distance to the watershed divide (in feet), y is the average watershed slope (per cent) and

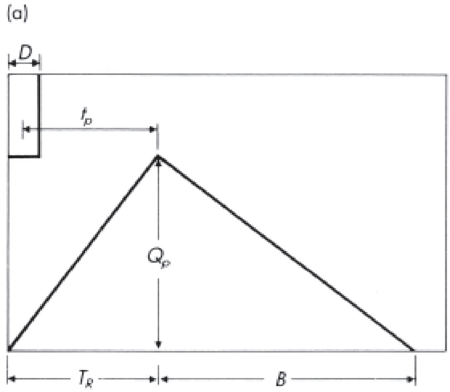


Figure II.5.12. SCS traiangular unit hydrograh

S and CN are obtained from Table II.5.7. The basin lag (t_p) is applicable to CN values between 50 and 95, and watershed areas less then 8 km^2. For urban areas, t_p should be adjusted for imperviousness. The coefficient 0.208 in equation 5.60 is an average value for many watersheds. It may be reduced by about 30 per cent for flat or swampy watersheds, or increased by about 20 per cent for steep basins. When such a change is introduced, then the unit hydrograph must also be adjusted accordingly.

Once Q_p and t_p are estimated, the unit hydrograph can be graphed and/or tabulated using the dimensionless unit hydrograph shown in Table II.5.9. Muzik and Chang (2003) developed a regional dimensionless hydrograph.

The SCS method is widely used (Byczkowski, 1996; Maidment, 1993) because of its simplicity, readily available watershed information, ease of application, and because it gives reasonable results. However, the results of studies comparing prediction with measured data have been mixed (Dingman, 2002) and so the method should be used with caution.

5.10.5 Flood hydrograph conceptual models

Recent advances in computer technology and theoretical hydrological developments have revolutionized the manner in which computations are now routinely performed. Hydrologic models allow for parameter verification in space and time, use of remotely sensed data and application of geographical information systems. Advanced computer-based technologies such as spreadsheets, databases and graphical capabilities facilitate the flexibility of data entry procedures.

Some of the more widely used models that have also been developed include:

(a) HEC-1 which was developed and is maintained by the US Army Corps of Engineers Hydrologic Engineering Center (www.hec.usace.army.mil). This model simulates the watershed as a series of hydraulic and hydrological components and calculates runoff from single storms. The user can select from a variety of sub-models that simulate precipitation, infiltration and runoff, as well as a variety of techniques to perform the flow routing. The model also includes dam safety and failure analysis, flood damage analysis and parameter optimization. More recent improvements include consideration of radar rainfall as input and the use of geographical information system and mapping tools (HEC-GeoRAS) for handling output and data manipulation;

(b) SCS-TR 20 (for agricultural watersheds) and SCS-TR 55 (for urban watersheds) were developed and are maintained by the Natural Resources Conservation Service, US Department of Agriculture. This combined model uses a curve number (CN) method to calculate the runoff hydrograph resulting from a single storm from sub areas and routed through drainage systems and reservoirs;

(c) SWMM was developed and is maintained by the US Environmental Agency (www.epa.gov/cdnnrmrl/models/swmm). This model consists of a runoff module, a transport module and a storage/treatment module. It simulates runoff quantity and quality, routes sewer flows, computes hydraulic head and simulates the effects of detentions basins and overflows. It is the most comprehensive model for handling urban runoff.

There are certainly many other good models that can perform the same tasks. Model capabilities change rapidly and therefore it is advisable to seek

Table II.5.9. Ordinates of the Natural Resources Conservation Service Dimensionless Unit Hydrograph

t/T_R	0	0.4	0.8	1.2	1.6	2.0	2.4	2.8	3.4	4.6	5.0
Q/Q_p	0	0.310	0.930	0.930	0.560	0.280	0.147	0.077	0.029	0.003	0.000

current information through the Websites of various model developers. Links to other popular models are www.wallingfordsoftware.com, www.dhi.dk, http://water.usgs.gov/software/lists/surface_water and www.haested.com.

All of the above models can be run on microcomputers and some are proprietary. Bedient and Huber (2002) provided a more comprehensive list of many internet sources to operational computer models, but many more have certainly become available in the intervening years.

5.10.6 Snowmelt contribution to flood

In some regions of the world, floods are caused by a combination of snowmelt and rainfall runoff or snowmelt alone. Factors affecting the contribution of snowmelt to floods include accumulated snow pack depth at time of melt, ice jamming, basin storage and the return period of the event in question. Synthesis of runoff hydrographs associated with snowmelt requires empirical equations, since snowmelt is not measured directly.

After the depth of melt has been estimated, it can be treated like rainfall and converted into streamflow by application of the unit hydrograph or routing technique. Such a procedure does not provide the probability of occurrence of a flood. Comparison of several snowmelt runoff models is described by WMO (1986*b*). There are several operational models that have a snowmelt routine, including HEC-1 (USACE, 1985).

5.10.7 Calculating discharges from urban drainage systems

Urban hydrology is concerned mainly with the prediction of runoff peaks, volumes and complete hydrographs anywhere in the system. The solution to the above problems requires various analytical methods. Peak volumes can be obtained from simplified techniques such as the rational method (see 5.10.4.2.1), while hydrographs usually require more comprehensive analysis including the Natural Resources Conservation Service method (see 5.10.4.2.2), or computer models (see 5.10.5). Urban drainage is discussed in more detail in 4.7.

5.10.8 Risk

The probability that the design flood will be exceeded at least once is known as the risk of failure, and the probability that the design flood will not be exceeded is referred to as the reliability. One of the main concerns in design-flood synthesis is an evaluation of the risks associated with the occurrence of floods higher than the design flood. Knowledge of these risks is important because of their social, environmental and economic implications, for example in the determination of flood-insurance rates, flood-zoning policies or water quality conservation. As floods are stochastic phenomena, their magnitude and the time of their future occurrence cannot be predicted. The only possibility is to assess them on a probabilistic basis, that is, to assign a probability to the possibility that a flood of a given magnitude will be exceeded within a specific period of time. A variable that has a probability of exceedance p has a return period $T = 1/p$.

Guidance for general frequency analysis is provided in 5.3, and in 5.9 for flood frequency analysis. A comprehensive risk assessment procedure for natural hazards is provided in *Comprehensive Risk Assessment for Natural Hazards* (WMO/TD-No. 955).

The probability of exceedance of a given magnitude of event, as derived from a probability distribution model, pertains to each future event. Thus, if an annual flood series is considered, the exceedance probability p defines the risk that the given magnitude will be exceeded in any one year. However, it is often necessary to calculate a probability p_n that a given event, for example the exceedance of a particular flood peak, will occur at least once in n years, for example, during the design life of a project. If the assumption of independence of floods in individual years is satisfied, this probability is:

$$p_n = 1 - (1 - p)^n = 1 - \left(1 - \frac{1}{T}\right)^n \qquad (5.63)$$

where T is the return period. This measure of risk provides a more probabilistic indication of the potential failure of the design than that encapsulated in the concept of return period. Note that the risk of an event occurring at least once during its return period follows from equation 5.63 for n equal to T. When T is large, this risk approaches the asymptotic value:

$$1 - e^{-1} = 0.63 \qquad (5.64)$$

From equation 5.63, it is possible to express T as a function of n and p_n, that is, to calculate a return period such that the risk of occurrence of the event during a period of n years will have a specified value p_n. This return period is called the design return period T_d and is as follows:

$$T_d = 1/[1 - (1 - p_n)^{1/n}] \qquad (5.65)$$

Table II.5.10. Required design return period T_d of an event whose risk of occurrence in *n* years is equal to p_n

	n year			
p_n	*2*	*10*	*50*	*100*
0.01	199.0	995.0	4975.0	9950.0
0.10	19.5	95.4	475.0	950.0
0.50	3.4	14.9	72.6	145.0
0.75	2.0	7.7	36.6	72.6

Some values of the variables T_d, n, and p_n are shown in Table II.5.10. In order to illustrate its use, assume that the design life of a dam is 50 years and that the designer wishes to take only a 10 per cent risk that the dam will be overtopped during its design life. Thus n equals 50, p_n equals 0.10, and the dam must be designed to withstand a flood that has a return period T_d of 475 years, which gives a probability of exceedance $p = 1/Td \approx 0.2$ per cent.

References and further reading

Adamowski, K., 1981: Plotting formula for flood frequency, *Water Research Bulletin*, Vol. 17, No. 2.

Adamowski, K., 1985: Nonparametric kernel estimation of flood frequencies, *Water Resources Research*, 21(11): 1585–1590.

Adamowski, K., 1989: A Monte Carlo comparison of parametric and nonparametric estimation of flood frequencies, *Journal of Hydrology*, 108: 295–308.

Adamowski, K., 1996: Nonparametric Estimation of Low-Flow Frequencies, *ASCE Journal of Hydraulic Engineering*, 122: 46–49.

Adamowski, K., Y. Alila and P.J. Pilon, 1996: Regional rainfall distribution for Canada, *Atmospheric Research*, 42, 75–88.

Akaike, H. 1974: A New Look at the Statistical Model Identification. IEEE transactions on automatic control, AC-19(6): 716–723.

American Society of Civil Engineers,1996: H*ydrology Handbook*, Second Edition, ASCE Manual and Reports on Engineering Practice No. 28, New York, 784 pp.

Arihood, L.D. and D.R. Glatfelter, 1986: *Method for estimating low-flow characteristics for ungaged streams in Indiana*, US Geological Survey Open-File Report 86–323.

Arihood, L.D. and D.R. Glatfelter, 1991: *Method for estimating low-flow characteristics for ungaged streams in Indiana*, US Geological Survey Water-Supply Paper 2372, 18 pp.

Arnell, N.W., 2003: Relative effects of multi-decadal climate variability and change in the mean and variability of climate due to global warming: future streamflows in Britain, Journal of Hydrology 270, 195–213.

Arnell, N.W., C. Liy, R. Compagnucci, L. da Cunha, K. Hanaki, C. Howe, G. Mailu, I. Shikomanov, E. Stakhiv, Hydrology and Water Resources, In: J.J. McCarthy, O. Canziani, N.A. Leary, D.J. Dokken and K.S. White (eds.), 2001: Climate Change 2001: Impacts, Adaptations, and Vulnerability. Contribution of the Working Group II to the Third Assessment Report of the Intergovernmental Panel on Climate Change, Cambridge University Press, Cambridge, pp. 191–233.

Arnell, V., P. Harremoes, M. Jensen, N.B. Johansen and J. Niemczynowicz, 1984: *Review of rainfall data application for design and analysis,Water Science and Techn.*, 16, pp. 1–45.

Asselin, J., T.B.M.J. Ouarda, V. Fortin and B. Bobée, 1999: *Une procedure Bayésienne bivariée pour détecter un décalage de la moyenne*. INRS-Eau, Rapport de Recherche R-528, 35 pp.

Bedient, P. B. and W. C. Huber, 2002: *Hydrology and Floodplain Analysis*, Prentice Hall, Inc. New Jersey, USA.

Benson, M.A., 1962: Factors affecting the occorance of floods in the south-west. US Geological Survey Water-Supply Paper 1580-D, Reston, Virginia.

Beran, M., J.R.M. Hosking and N. Arnell, 1986: Comment on "Two-Compent Extrreme Value Distribution for Flood Frequency Analysis, Water Resources Research 220(2), 263–266.

Bingham, R.H., 1982: *Low-flow characteristics of Alabama streams*, US Geological Survey Water-Supply Paper 2083, 27 pp.

Bobée, B. and F. Ashkar, 1991: *The gamma family and derived distributions applied in Hydrology*. Water Resources Publications, Littleton, CO, 203 pp.

Box, G.E.P. and G.C. Tiao, 1973: Bayesian inference in statistical analysis, Addison-Wesley, Reading, MA.

Bras, R. and I. Rodriguez-Iturbe, 1985: *Random functions in hydrology*. Addison-Wesley, Reading, Massachusetts.

Burn, D.H., 1990: Evaluation of regional flood frequency analysis with a region of influence approach. *Water Resources Research*, 26(10): 2257–2265.

Byczkowski, A, 1996: *Hydrology*, (in Polish), Wydawnictwo SGGW (Poland.)

Carpenter, D.H. and D.C. Hayes, 1996: Low-flow characteristics of streams in Maryland and Delaware, US Geological Survey Water-Resources Investigations Report 94–4020, 113.

Carswell, W.J. and S.V. Bond, 1980: Multiyear low flow of streams in northeastern Kansas, US Geological Survey Open-File Report 80–734, 26 pp.

Cavadias, G.S., 1990: The canonical correlation approach to regional flood estimation. In M. A. Beran, M. Brilly, A. Becker and O. Bonacci (eds.), *Proceedings*

of the Ljubljana Symposium on Regionalization in Hydrology, IAHS Publ. No. 191: 171–178. Wallingford, England.

Chow, V.T., 1964: *Handbook of Applied Hydrology*. McGraw-Hill, New York.

Chowdhury, J.U., J.R. Stedinger and L-H. Lu, 1991: Goodness-of-fit tests for regional GEV flood distributions, *Water Resources Research* 27(7), 1765–1776.

Clarke, R.T., 1994: *Statistical Modelling in Hydrology*, John Wiley and Sons, Chichester, England.

Cluckie, I.D., P.P. Ede, M.D. Owens and A.C. Bailey, 1987: Some Hydrological Aspects of Weather Radar Research in the United Kingdom, *Hydrological Sciences Journal*, 32,3,9, pp. 329–346.

Cluckie, I.D. and M.L. Pessoa 1990: Dam Safety: An Evaluation of Some Procedures for Design Flood Estimation, *Hydrological Sciences Journal*, 35(5), pp. 547–569.

Cohn, C.A., W.L. Lane and J.R. Stedinger, 2001: Confidence Intervals for EMA Flood Quantile Estimates, *Water Resources Research*, 37(6), 1695–1706.

Collier, C.G., 1993: Estimating Probable Maximum Precipitation (PMP) Using a Storm Model and Remotely-sensed Data, British Hydrological Society 4th National Hydrology Symposium, Cardiff, September 13–16, *Institute of Hydrology*, pp. 5.33–5.38.

Condie, R. and L.C. Cheng, 1982: *Low Flow Frequency Analysis Program LOFLOW*, Inland Waters Directorate, Water Resources Branch, Environment Canada.

Condie, R. and G.A. Nix, 1975: *Modelling of low flow frequency distributions and parameter estimation, presented at the International Water Resources Symposium*, Water for Arid Lands, Tehran, Iran, Dececmber 8–9.

Court, A.,1961: Area-depth rainfall formulas. Journal of Geophysical Research, Vol. 65, No. 6, June, pp. 1823–1831.

Cunnane, C., 1978: Unbiased plotting positions-A review, *Journal of Hydrology*, 37 (3/4).

Cunnane, C. 1988: Methods and merits of regional flood frequency analysis. *Journal of Hydrology*, 100: 269–290.

Cunnane, C. 1989: *Statistical Distributions for Flood Frequency Analysis*, Operational Hydrology Report No. 33, World Meteorological Organization WMO-No. No. 718, Geneva, Switzerland.

Dingman, S. L., 2002: *Physical Hydrology*, Prentice Hall, Inc. New Jersey, USA.

Draper, N. and Smith, H., 1981: *Applied Regression Analysis*, Second Edition, John Wiley & Sons, Toronto.

Durrans, S.R. and Tomic, S., 2001: Comparison of Parametric Tail Estimators for Low-Flow Frequency Analysis, *Journal of the American Water Resources Association*, 37(5), pp. 1203–1214.

England, J.F. Jr., J.D. Salas and R.D. Jarrett, 2003: Comparisons of two moments-based estimators that utilize historical and paleoflood data for the log Pearson type III distribution, *Water Resources Research* 39(9).

Fennessey, N.M. and R.M. Vogel, 1990: Regional Flow Duration Curves for Ungaged Sites in Massachusetts, *ASCE Journal of Water Resources Planning and Management*, 116(4), pp. 530–549.

Fernandez Mills, G.,1995: Principal component analysis of precipitation and rainfall regionalization in Spain, *Theoretical and Applied Climatology*, 50, 169–183.

Fill, H., and J. Stedinger, 1995: L–moment and PPCC Goodness-of-Fit tests for the Gumbel Distribution and Effect of Autocorrelation, *Water Resources Research*, 31(1), 225–29.

Fill, H. and J. Stedinger, 1998: Using regional regression within index flood procedures and an empirical Bayesian estimator, *Journal of Hydrology*, 210(1–4), 128–145.

Fortin, V., H. Perron, J. Bernier and B. Bobée, 1998: *Determination des crues de conception*, Rapport de recherché R-532, INRS-Eau, 103 pp.

Francés, F., 1998: Using the TCEV distribution function with systematic and non-systematic data in a regional flood frequency analysis, *Stochastic Hydrology and Hydraulics* 12 (4), 267–283.

Frederick, R.H., V.A. Meyers and E.P. Auciello, 1977: Five-to-60-minute precipitation frequency for the eastern and central United States, NOAA Technical Memorandum NWS HYDRO-35, *National Weather Service*, US Dept.of Commerce, Washington, D.C.

Gabriele, S. and N. Arnell, 1991: A Hierachical Approach to Regional Flood Frequency Analysis, *Water Resources Research*, 27(6), 1281–1289.

Gabriele, S. and P. Villani, 2002: The estimation of rainfall frequency in a Mediterranean environment due to extraordinary combinations of hydrological and climatic conditions, The Extremes of the Extremes: Extraordinary Floods (Proceedings of a symposium held at Reykjavik, Iceland, July 2000). IAHS Publ. No. 271, pp. 45–50.

Gingras, D., M. Alvo and K. Adamowski, 1995: Regional flood relationship by nonparametric regression, *Nordic Hydrology*, 26, 73–90.

Gray, D.M., ed., 1973: Handbook on the Principles of Hydrology, National Research Council of Canada, Water Information Center Inc., Syosset, N.Y..

Greenwood, J.A., J.M. Landwehr, N.C., Matalas and J.R. Wallis, 1979: Probability weighted moments: definition and relation to parameters of several distributions expressible in inverse form. *Water Resources Research*, Vol. 15, No. 5, pp. 1049–1054.

GREHYS, 1996*a*: Presentation and review of some methods for regional flood frequency analysis. *Journal of Hydrology*, 186(1–4): 63–84.

GREHYS, 1996*b*: Intercomparison of flood frequency procedures for Canadian rivers. *Journal of Hydrology*, 186(1–4): 85–103. F.E. Grubbs and G. Beck (1972). Extension of sample sizes and percentage points for significance tests of outlying observations. Technometrics, 14(4): 847–854.

Griffis, V.W., J.R. Stedinger and T.A. Cohn, 2004: LP3 Quantile Estimators with Regional Skew Information and Low Outlier Adjustments, accepted to Water Resources Research.

Grygier, J.C., J.R. Stedinger and H.B. Yin, 1989: A generalized move procedure for extending correlated series, *Water Resources Research*, 25(3), 345–350.

Guillot, P., 1993: The Arguments of the Gradex Method: a Logical Support to Assess Extreme Floods. Extreme Hydrological Events: Precipitation, Floods and Droughts. Proceedings of the Yokohama Symposium, IAHS Publ. No. 213.

Guo, S.L., 1991: Nonparametric variable kernel estimation with historical floods and paleoflood information, *Water Resources Research*, 27(1): 91–98.

Guo, S.L., R.K. Kachroo and R.J. Mngodo, 1996: Nonparametric Kernal Estimation of Low Flow Quantiles, *Journal of Hydrology*, 185, pp. 335–348.

Gumbel, E.J., 1958: *Statistics of extremes*, Columbia University Press, NY, 375 pp.

Gustard, A. and K.M. Irving, 1994: Classification of the Low Flow Response of European Soils, FRIEND: Flow Regimes from International Experimental and Network Data, IAHS Publication No. 221, pp. 113–117.

Hansen, E.M., L.C. Schreiner J.F. and Miller, 1982: Application of probable maximum precipitation estimates: United States east of the 105th meridian. Hydrometeorological Report, No. 52, US National Weather Service.

Helsel, D.R., and R.M. Hirsch, 1992: *Statistical Methods in Water Resources*, Elsevier, NY.

Herbst, P. H., K.B. Bredenkamp and H.M.G. Barker, 1966: A technique for the evaluation of drought from rainfall data. *Journal of Hydrology*, Vol. 4, No. 3, pp. 264–272.

Hershfield, D.M., 1961: Rainfall frequency atlas of the United States for durations from 30 minutes to 24-hours and return periods from 2 to 100 years, Technical Paper 40, US Weather Bureau, Washington, D.C.

Hershfield, D.M., 1965: Method for estimating probable maximum rainfall. *Journal of the American Waterworks Association*, Vol. 57, August, pp. 965–972.

Hirsch, R.M., 1982: A comparison of four record extension techniques, *Water Resources Research*, 18(4), 1081–1088.

Hirsch, R. M, and J. R. Stedinger,1987: Plotting positions for historical floods and their precision, *Water Resources Research*, 23(4), 715–727.

Hirschboeck, K.K., L.L. Ely and R.A. Maddox, 2000: Hydroclimatology of Meteorologic Floods, Chapter 2, Inland Flood Hazards: Human, Riparian and Aquatic Communities, E.E. Wohl (ed.), Cambridge University Press, Stanford, United Kingdom.

Hogg, W.D. and D.A. Carr, 1985: *Rainfall frequency atlas for Canada*, Canadian Government Publishing Centre, Ottawa, Canada.

Hosking, J.R.M., 1990: L–moments: Analysis and estimation of distributions using linear combinations of order statistics, *Journal of Royal Statistical Society*, B, 52(2), 105–124.

Hosking, J.R.M. and J.R. Wallis, 1987: Parameter and quantile estimation for the generalized Pareto distribution, *Technometrics*, 29(3):339–349.

Hosking, J.R.M. and Wallis, J.R., 1988, The effect of intersite dependence on regional flood frequency analysis. *Water Resources Research*, Vol. 24, No. 4, pp. 588–600.

Hosking, J.R.M. and J.R. Wallis, 1995: A comparison of unbiased and plotting-position estimators of L moments, *Water Resources Research*, 31(8), 2019–25.

Hosking, J.R.M. and J.R. Wallis, 1997: Regional Frequency Analysis: An Approach Based on L–moments, Cambridge University Press.

Hosking, J.R.M., J.R. Wallis and E.F. Wood, 1985: Estimation of the generalized extreme-value distribution by the method of probability weighted moments, *Technometrics* 27(3), 251–261.

Huff, F.A. and S.A., Jr. Changnon, 1964: A model 10-inch rainstorm, *Journal of Applied Meteorology*, pp. 587–599.

Institute of Hydrology, 1980: *Low Flow Studies, Research Report*, Wallingford, Oxon, UK.

Institute of Hydrology, 1999: *Flood Estimation Handbook*, CEH, Wallingford, UK.

Interagency Advisory Committee on Water Data, 1982: *Guidelines for Determining Flood Flow Frequency*, Bulletin #17B, US Department of the Interior, US Geological Survey, Office of Water Data Coordination, Reston Virginia.

Jain, S. and U. Lall, 2001: Floods in a changing climate: Does the past represent the future?, *Water Resources Research* 37(12), 3193–3205.

Javelle P., 2001: *Caractérisation du régime des crues: le modèle débit-durée-fréquence convergent*. Approche locale et régionale, PhD thesis.Cemagref-Lyon. Institut National Polytechnique de Grenoble. 268 pp. can be downloaded at: http://www.lyon.cemagref.fr/doc/these/javelle.

Javelle, P., T.B.M.J. Ouarda and B. Bobée, 2003: Flood regime definition using the flood-duration-frequency approach: Application to the Provinces of

Quebec and Ontario, Canada. *Hydrological Processes*. 17(18): 3717–3736.

Javelle, P., T.B.M.J. Ouarda, M. Lang, B. Bobée, G. Galea and J.-M. Gresillon, 2002: Development of regional flow-duration-frequency curves based on the index-flood method, *Journal of Hydrology*. 258: 249–259.

Kite, G.W., 1988: Frequency and Risk Analysis in Hydrology, *Water Resources Publication*, Littleton, CO.

Kottegoda, M. and R. Rosso, 1997: *Statistics Probability, and Reliability for Civil and Environmental Engineers*, McGraw-Hill, New York.

Kroll, C.N. and J.R. Stedinger, 1996: Estimation of Moments and Quantiles Using Censored Data, *Water Resources Research*, 32(4), pp. 1005–1012.

Kuczera, G., 1982: Combining site-specific and regional information: an empirical Bayes Approach. *Water Resources Research*, 18(2):306–314.

Kundziewicz Z.W., D. Rosbjerg, S.P. Simonowicz and K. Takeuchi, 1993: Extreme Hydrological Events: Precipitation, Floods and Droughts, IAHS Press, *Institute of Hydrology*, UK, Publication No. 213.

Landwehr, J.M., N.C. Matalas and J.R. Wallis, 1979: Probability weighted moments compared with some traditional techniques in estimating Gumbel parameters and quantiles, *Water Resources Research*, 15(5), 1055–-1064.

Lang, M., Ouarda, T.B.M.J. and B. Bobée, 1999: Towards operational guidelines for over-threshold modeling. *Journal of Hydrology*, 225: 103–117.

Lawal, S.A. and W.E. Watt, 1996: Non-Zero Lower Limit in Low Flow Frequency Analysis?, *Water Resources Bulletin*, 32(6), pp. 1159–1166.

Lettenmaier, D.P. and K.W. Potter, 1985: Testing flood frequency estimation methods using a regional flood generation model. *Water Resources Research*, Vol. 21, No. 12, pp. 1903–1914.

Lettenmaier, D.P., J.R. Wallis and E.F. Wood, 1987: Effect of regional hereogeneity on flood frequency estimation, *Water Resources Research*, 23(2), 313–324.

Madsen, H., and D. Rosbjerg, 1997, Generalized least squares and empirical Bayes estimation in regional partial duration series index-flood modeling, *Water Resources Research*, 33(4), 771–782.

Maidment, D. R. (editor in chief), 1993: *Handbook of Hydrology*, McGraw-Hill, Inc., New York, USA.

Mann, H.B. and D.R. Whitney, 1947: On the test of whether one of two random variables is stochastically larger than the other. *Annual Mathematical Statistics*, 18: 50–60.

Martins, E.S. and J.R. Stedinger, 2000: Generalized Maximum Likelihood Pareto-Poisson Flood Risk Analysis for Partial Duration Series, *Water Resources Research* 37(10), 2559-2567, 2001.

Martins, E.S. and J.R. Stedinger, 2001: Historical information in a GMLE-GEV Framework with Partial Duration and Annual Maximum Series, *Water Resources Research*, 37(10), 2551–2557.

Matalas, N.C. and B. Jacobs, 1964: *A correlation procedure for augmenting hydrologic data*, US Geol. Prof. Paper, 434-E, E1-E7.

Matalas, N., 1963: *Probability Distributions of Low Flows*, Professional Paper 434-A, USGS, Washington D.C.

Miller, J.F. 1972: *Physiographically Adjusted Precipitation-frequency Maps: Distribution of Precipitation in Mountainous Areas*. WMO-No. 326 (11): 264–277.

Miller, J.R., R.H. Frederick and R.J. Tracey, 1973: Precipitation frequency atlas of the western United States, NOAA Atlas 2, National Weather Service, NOAA, US Department of Commerce, Silver Spring, MD.

Moore, R., 1993: *Applications of Weather Radar Data to Hydrology and Water Resources*, WMO, Regional Association VI, (Europe), Working Group on Hydrology, 26 pp.

Morlat G., 1956: *Les lois de probabilité de Halphen. Revue de statistique appliqué*, 3: 21-43. (http://archive.numdam.org/ARCHIVE/RSA/RSA_1956__4_3/RSA_1956__4_3_21_0/RSA_1956__4_3_21_0.pdf).

Muirhead, R.J., 1982: *Aspect of Multivariate Statistical Theory*. J. Wiley, 673 pp.

Muzik,I. and C. Chang, 2003: Regional dimensionless hydrograph for Alberta foothills, *Hydrological Processes*, 17, 3737–3747.

Nathan, R.J. and T.A. McMahon, 1990: Practical Aspects of Low-Flow Frequency Analysis, *Water Resources Research*, 26(9), pp. 2135–2141.

Nathan, R.J. and T.A. McMahon, 1992: Estimating Low Flow Characteristics in Ungauged Catchments, *Water Resources Management*, 6, pp. 85–100.

National Research Council, 1988: *Estimating probabilities of extreme floods*, National Academy Press, Washington, D.C., 141 pp.

National Research Council of Canada (NRCC), 1989: *Hydrology of floods in Canada: a guide to planning and design*, W.E.Watt editor, Ottawa, 245 pp.

Natural Environment Research Council, 1975: Flood Studies Report, Volume II: Meteorological Studies, London, England, 81 pp.

NERC, 1975: Flood Studies Report, Vol. 1, Hydrological Studies, Natural Environment Research Council, 27 Charing Cross Road, London.

Nguyen,V-T-V., N. In-na and B. Bobee, 1989: A new plotting position formula for Pearson type III distribution, *ASCE Journal of Hydraulic Engineering*, 115(6), pp. 709–730.

Nguyen, V-T-V., T-D. Nguyen and F. Aaskar, 2002: Regional Frequency Analysis of Extreme Rainfalls, *Water Science and Technology*, Vol. 45, No. 2, pp. 75–81.

Nguyen,V-T-V., J. Rousselle and M.B. McPherson, 1981: Evaluation of areal versus point rainfall with sparse

data, *Canadian Journal of Civil Engineering*, 8(2), pp. 173–178.

Niemczynowicz, J. 1982: Areal intensity-duration-frequency curves for short term rainfall events, *Nordic Hydrology*, 13(4), pp. 193–204.

NRC, 1998: Decade-to-Century-Scale Climate Variability and Change: A Science Strategy. National Resources Council, Panel on Climate Variability on Decade-to-Century Time Scales, Washington, DC: National Academy Press.

NRCS, 1972: National Engineering Handbook, Section 4, "Hydrology," Chapter 9.

Olsen, J.R., J.R. Stedinger, N.C. Matalas and E.Z. Stakhiv, 1999. Climate variability and flood frequency estimation for the Upper Mississippi and Lower Missouri Rivers, *Journal of the American Water Resources Association*, 35(6), 1509–1524.

Ouarda, T.B.M.J., G. Boucher, P.F. Rasmussen and B. Bobée, 1997: *Regionalization of floods by canonical analysis*. In "Operational Water Management", J.C. Refsgaard and E.A Karalis Eds., A.A. Balkema Pub. Rotterdam, 297–302.

Ouarda, T.B.M.J., M. Haché and B. Bobée, 1998: *Régionalisation des événements hydrologiques extrêmes*, Research Report No. R-534, INRS-Eau, University of Quebec, Ste-Foy, Canada, 181 pp.

Ouarda, T.B.M.J., M. Lang, B. Bobée, J. Bernier, P. Bois, 1999: Analysis of regional flood models utilized in France and Quebec, *Review Science Eau*, 12(1): 155–182.

Ouarda, T.B.M.J., M. Haché, P. Bruneau and B. Bobée, 2000: Regional flood peak and volume estimation in northern Canadian basin. *Journal of Cold Regions Engineer*, ASCE. 14(4): 176–191.

Ouarda, T.B.M.J., C. Girard, G.S. Cavadias and B. Bobée. 2001: Regional flood frequency estimation with canonical correlation analysis. *Journal of Hydrology* 254: 157–173.

Ozga-Zielinska, M.J. Brzezinski and B. Ozga-Zielinski, 1999: *Calculation of Design Flood for Hydraulic Structures*, Institute of Meteorology and Water Resources, No. 27 (in Polish).

Ozga-Zielinski B., 2002: The Gradex-KC and Gradex-ZN Methods for Estimation of Maximum Floods with T-year Return Period for Cases with Short or Nonexisting Flow Observations. Monographs of Water Management Committee of the Polish Academy of Sciences, No.21 (in Polish).

Pandey,G. R. and V.T.V. Nguyen. 1999: A comparative study of regression based methods in regional flood frequency analysis. *Journal of Hydrology*. 225: 92–101.

Paulson, R.W., E.B. Chase, R.S. Roberts and D.W. Moddy, 1991: National Water Summary 1988–89 – Hydrologic Events and Floods and Droughts, US Geological Survey Water.

Pekarova, P., P. Miklanek and J. Peklar, 2003: Spatial and temporal runoff oscillation analysis of the main rivers of the world during the 19th and 20th centuries, *Journal of Hydrology*, 274, 62–79.

Perreault L., B. Bobée and P.F. Rasmussen, 1999*a*: Halphen distribution system, I: Mathematical and statistical properties. *Journal of Hydrologic Engineering*, ASCE, 4(3): 189–199.

Perreault L., B. Bobée and P.F. Rasmussen, 1999*b*: Halphen distribution system, II: Parameter and quantile estimation. *Journal of Hydrologic Engineering*, ASCE, 4(3): 200–208.

Perreault, L., M. Haché, M. Slivitzky and B. Bobée, 1999*c*: Detection of changes in precipitation and runoff over eastern Canada and US using a Bayesian approach, Stoch. Env. Resear. And Risk Assess. 13: 201–216.

Perreault, L, J. Bernier, B. Bobée and E. Parent, 2000: Bayesian change-point analysis in hydrometeorological time series, Part 1, The normal model revisited, *Journal of Hydrology*. 235: 221–241.

Piechota, T.C. and J. A. Dracup, 1999: Long-range streamflow forecasting using El Nino-Southern Oscillation indicators, *Journal of Hydrologic Engineering*, ASCE, 4(2), 144–151.

Pilgrim D.M. and I. Cordery, 1993: Flood Runoff, in Handbook of Hydrology, editor Maidment D. R., McGraw-Hill, Inc., New York, USA.

Pilgrim, D.H. and D.G. Doran, 1993: Practical Criteria for the Choice of Method for Estimating Extreme Design Floods, IAHS Publication No. 213, Institute of Hydrology, Wallingford, UK.

Pilgrim, D. and Cordery, I., 1975: Rainfall temporal patterns for design floods, ASCE *Journal of Hydraulic Engineering*, 101 (HY1), pp. 81–95.

Pilgrim, D.H., ed., 1998: *Australian Rainfall and Runoff: A Guide to Flood Estimation*, Volumes I and II, Institution of Engineers Australia, Canberra.

Pilon, P.J. and K.D. Harvey, 1992: The Consolidated Frequency Analysis (CFA) Package. (HOMS component I.81.2.02), Environment Canada, Water Resources Branch, Ottawa.

Pilon, P.J. and S.Yue, 2002: Detecting climate-related trends in streamflow data, *Water Science and Technology*, 45(8), 89–104.

Porparto, A. and L. Ridolfi, 1998: Influence of weak tends on exceedance probability, *Stochastic Hydrology and Hydraulics* 12(1), 1–15.

Reilly, C.F. and C.N. Kroll, 2003: Estimation of 7-day, 10-year low-streamflow statistics using baseflow correlation, *Water Resources Research*, 39(9), 1236.

Ries, K.G., 1994: Development and application of Generalized least squares regression models to estimate low-flow duration discharges in Massachusetts, US Geological Survey Water-Resources Investigations Report 94–4155, 33 pp.

Robson, A. and D. Reed, 1999: Flood Estimation Handbook, Volume 3, Statistical procedures for

flood frequency estimation, *Institute of Hydrology*, Wallingford, Oxfordshire, United Kingdom.

Rossi, F., M. Fiorentino and P. Versace, 1984: Two-Component Extreme Value Distribution for Flood Frequency Analysis, *Water Resources Research* 20(7), 847–856.

Salas, J.D., E.E. Wold and R.D. Jarrett,1994: Determination of flood characteristics using systematic, historical and paleoflood data, in G. Rossi et al., Coping with Floods, pp. 111–134, Kluwer Academic Publishers, Netherlands.

Schaefer, M.G., 1990: Regional analysis of precipitation annual maxima in Washington State, *Water Resources Research*, 26(1), pp. 119–131.

Schwarz, G., 1978: Estimating the dimension of a model. *The Annals of Statistics*, 6: 461–464.

Searcy, J.K., 1959: *Flow-Duration Curves*, US Geological Survey Water Supply Paper 1542–A.

Sevruk, B., 1985: Correction of precipitation measurements: summary report, Workshop on the correction of precipitation measurements, April 1–3, 1985, Zurich, Switzerland, pp. 13–23

Sherwood, J.M., 1994: Estimation of volume-duration-frequency relations of ungauged small urban streams in Ohio. *Water Resources Bulletin* 302: 261–269.

Singh, V. P., 1992: *Elementary Hydrology*, Prentice-Hall Inc., Englewood Cliffs, NJ 07632.

Smakhtin,V.U., 2001: Low Flow Hydrology: A Review, *Journal of Hydrology*, 240, pp. 147–186.

Smith, J.A. 1993: *Precipitation, in Handbook of Hydrology,* D.R. Maidment, ed., McGraw-Hill, New York, 3.1–3.47.

Stedinger, J.R. and V.R. Baker, 1987: Surface water hydrology: historical and paleoflood information, US National report to international Union of Geodesy and Geophysics, 1983–1987, R*eviews of Gyophsysical and Space Physics*, 25(2), 119–124.

Stedinger J.R., R.M. Vogel and E. Foufoula-Georgiou, 1993: Frequency analysis of extreme events. Chapter 18 (1–66) in: *Handbook of Applied Hydrology*; D.R. Maidment (Ed.) McGraw-Hill, NY.

Stedinger, J.R. and L. Lu, 1995: Appraisal of Regional and Index Flood Quantile Estimators, *Stochastic Hydrology and Hydraulics*, 9(1), 49–75.

Stedinger,J.R. and T.A. Cohn, 1986, Flood frequency analysis with historical and paleoflood information, *Water Resources Research*, 22(5):785–793.

Stedinger, J.R. and G.D. Tasker, 1985: Regional Hydrologic Analysis 1. Ordinary, Weighted, and Generalized Least Squares Compared, *Water Resources Research*, 21(9), pp. 1421–1432.

Stedinger, J.R., 1980: Fitting Log Normal Distributions to Hydrologic Data, *Water Resources Research*, 16(3), 481–490.

Stedinger, J.R., G. Tasker, 1986: Regional hydrological analysis, 2, model-error estimators, estimation of sigma and log Pearson type 3 distributions. *Water Resources Research*, 22(10):1487–1499.

Stedinger, J.R. and W.O. Thomas, Jr., 1985: *Low-flow estimation using base-flow measurements*, US Geological Survey Open-File Report 85–95, 22 pp.

Task Committee of the Hydraulics Division, 1980: Characteristics of Low Flows, ASCE *Journal of the Hydraulics Division*, 106 (HY5), pp. 717–737.

Tasker, G.D., 1980: Hydrologic Regression with Weighted Least Squares, *Water Resources Research*, 16(6), pp. 1107–1113.

Tasker, G.D., 1987: A Comparison of Methods for Estimating Low Flow Characteristics of Streams, *Water Resources Bulletin*, 23(6), pp. 1077–1083.

Tasker, G.D., 1991: Estimating low-flow characteristics of streams at ungaged sites, US Geological Survey Open-File Report 91–244, Proceedings of the United States – People's Republic of China Bilateral Symposium on Droughts and Arid-Region Hydrology, September 16–20, 1991, Tucson, Arizona, pp. 215–219.

Terry, M.E. 1952: Some rank order tests which are most powerful against specific parametric alternatives. *Ann. Math. Statist.*, 23 : 346–366.

Thomas, W.O., 1985: A uniform technique for flood frequency analysis, *Journal of Water Resources*. Planning and Management, 111(3), 321–337.

Thomas, D.M. and M.A. Benson, 1970: Generalization of Streamflow Characteristics from Drainage-Basin Characteristics, US Geological Survey Water-Supply Paper 1975, Reston, Virginia.

Thomas, W.O., Jr. and J.R. Stedinger, 1991: Estimating low-flow characteristics at gaging stations and through the use of base-flow measurements, US Geological Survey Open-File Report 91-244, pp. 197–206.

US Army Corps of Engineers, 1985: HEC-1 Flood Hydrograph Package, *Hydrologic Engineering Center*, Davis, California.

Viessman, W. and G.L.Lewis, 2003: *Introduction to Hydrology*, Prentice Hall, Pearson Education, Inc., Upper Saddle River, NJ 07458, USA.

Vogel, R. and J.R. Stedinger, 1984: Flood plain delineation in ice jam prone regions, Journal of the Water Resources Planning and Management, ASCE, 110(WR2), 206–219.

Vogel, R.M. and N.M. Fennessey, 1994: Flow Duration Curves I: A New Interpretation and Confidence Intervals, ASCE *Journal of Water Resources Planning and Management*, 120(4), pp. 485–504.

Vogel, R.M. and N.M. Fennessey, 1995: Flow Duration Curves II: A Review of Applications in Water Resources Planning, *Water Resources Bulletin*, 31(6), pp. 1029–1039.

Vogel, R.M. and C.N. Kroll, 1989: Low-Flow Frequency Analysis Using Probability-Plot Correlation Coefficients, ASCE *Journal of Water*

Resources Planning and Management, 115(3), pp. 338–357.

Vogel, R.M. and C.N. Kroll, 1990: Generalized Low-Flow Frequency Relationships for Ungaged Sites in Massachusetts, *Water Resources Bulletin*, 26(2), pp. 241–253.

Vogel, R.M. and C.N. Kroll, 1992: Regional Geohydrologic-Geomorphic Relationships for the Estimation of Low-Flow Statistics, *Water Resources Research*, 28(9), pp. 2451–2458.

Vogel, R.M., and J.R. Stedinger, 1985: Minimum variance streamflow record augmentation procedures, *Water Resources Research*, 21(5), 715–723.

Vogel, R.M., and N.M. Fennessey, 1993. L–moment diagrams should replace product moment diagrams, *Water Resources Research*, 29(6), 1745–52.

Wald, A. and J. Wolfowitz, 1943: An exact test for randomness in the non-parametric case based on serial correlation. *Annual Mathematical Statistics*, 14: 378–388.

Waltemeyer, S.D., 2002: Analysis of the Magnitude and Frequency of the 4-Day Annual Low Flow and Regression equations for Estimating the 4-Day, 3-Year Low-Flow Frequency at Ungaged Sites on Unregulated Streams in New Mexico, US Geological Survey, Water-Resources Investigations Report 01–4271, Albuquerque, New Mexico.

Wang, Q.J., 1991: The POT model described by the generalized Pareto distribution with Poisson arrival rate, *Journal of Hydrology*, 129, pp. 263–280.

Wang, Q.J., 1996: Direct sample estimators of L–moments, *Water Resources Research*, 32(12), 3617–19.

Waylen, P. and M-K. Woo, 1982: Prediction of annual floods generated by mixed processes, *Water Resources Research*, 18(4), 1283–86.

Waylen, P.R. and M.-K. Woo, 1987: Annual Low Flows Generated by Mixed Processes, *Hydrological Sciences Journal*, 32(3), pp. 371–383.

Weisberg, S., 1980: *Applied Linear Regression*, John Wiley & Sons, Toronto

Weisner, C.J., 1970: *Hydrometeorology*, Chapman & Hall, London.

Wenzel, H.G., 1982: Rainfall for urban stormwater design, in *Urban Storm Water Hydrology*, D.F. Kibler, ed., Water Resources Monograph No. 7, American Geophysical Union, Washington, D.C.

Wilks, D.S. 1993: Comparison of the three-parameter probability distributions for representing annual extreme and partial duration precipitation series, *Water Resources Research*, 29(10), pp. 3543–549.

Wohl, E.E. (ed.), 2000: *Inland flood hazards: human, riparian and aquatic communities*, Cambridge University Press, Stanford, UK.

World Meteorological Organization, 1969*a*: *Estimation of Maximum Floods*, WMO No. 233, TP 126, Technical Note No. 98, Geneva, Switzerland.

———, 1969*b*: *Manual for Depth-area-duration Analysis of Storm Precipitation*, WMO-No. 237, Geneva.

———, 1986*a*: *Manual for Estimation of Probable Maximum precipitation*, Second Edition, Operational Hydrology Report No. 1, WMO-No. 332, Geneva, 269 pp.

———, 1986*b*: *Intercomparison of models of snowmelt runoff*, WMO-No.646, Operational Hydrology Rep. No. 23, Geneva.

———, 1981: Selection of Distribution Types for Extremes
of Precipitation (B. Sevruk and H. Geiger). *Operational Hydrology Report No. 15*, WMO-No. 560, Geneva.

———, 1999: *Comprehensive Risk Assessment for Natural Hazards*, WMO/TD-No. 955.

WRC, 1967: Guidelines for Determining flood flow frequency. US Water Resources Council Hydrology Committee, Washington, D.C. Revised versions: Bulletin 15(1975) – 17(1976) – 17 A (1977) – 17B (1981).

Yue, S., P. Pilon and G. Cavadias, 2002*a*: Power of the Mann-Kendall and Spearman's rho tests for detecting monotonic trends in hydrological series, *Journal of Hydrology*, 259, 254–271.

Yue, S., P. Pilon, B. Phinney and G.Cavadias, 2002*b*: The influence of autocorrelation on the ability to detect trend in hydrological series, *Hydrological Processes*, 16, 1807–1829.

Yue, S. and P. Pilon, 2004: A comparison of the power of the t test, Mann-Kendall and bootstrap
tests for trend detection, *Hydrological Sciences*, 49(1), 21–37.

Zaidman, M.D., V. Keller, A.R. Young and D. Cadman, 2003: Flow-Duration Frequency Behaviour of British Rivers Based on Annual Minima Data, *Journal of Hydrology*, 277, pp. 195–213.

Zalina, M.D., Desa, M.N.M., Nguyen, V-T-V., and Amir, K. 2002, Statistical Analysis of Extreme Rainfall Processes in Malaysia, *Water Science and Technology*, Vol. 45, No. 2, pp. 63–68.

CHAPTER 6

MODELLING OF HYDROLOGICAL SYSTEMS

The general term modelling means replacing an object under consideration by a quasi-object, or model, in order to draw information about the object from the model. The model imitates or mimics selected aspects of the object of interest, which are deemed important in the study at hand. A model can be seen as a working analogy of the real object, allowing similarity, though not identity, of properties, which are important in the particular statement of the problem. The basic rationale of modelling is the possibility to simulate and predict the behaviour of a complex object or system, with the help of a simpler, and/or more tractable model. Details of the real object can be ignored because they are not important in a particular case or because they are too complex, hence not tractable (see Dooge, 1973).

Many ways of classifying models have been proposed, starting from the early distinction of intuitive and formalized models. Formalized models can be divided into material models and symbolic models. The class of material models, the representation of a real system by another real system, can be divided into physical models, also called iconic, or look-alike models, such as hydraulic laboratory models of a dam or a channel built to an appropriate scale, and analogue models, such as electrical analogs. Material models have similar properties to the object under consideration and are easier and cheaper to study. Experiments on material models can be made under more favourable and observable conditions (Singh, 1988), while experiments on the object may be difficult or even impossible. Symbolic models can be classified into verbal, graphical and mathematical models. Nowadays, mathematical models are by far the most commonly used, mainly because of the computational capabilities offered by affordable computers.

The notion of the mathematical modelling of hydrological systems can be understood in a very broad sense as the use of mathematics to describe features of hydrological systems or processes. Hence, every use of a mathematical equation to represent links between hydrological variables, or to mimic a temporal or spatial structure of a single variable, can be called mathematical modelling. Under such a broad definition of the term, there are multiple links to many chapters of the Guide, as every hydrological process can be described via mathematical formalisms. The term mathematical modelling of hydrological systems includes time-series analysis and stochastic modelling, where the emphasis is on reproducing the statistical characteristics of a hydrological times series of a hydrological variable.

Developments in the modelling of hydrological systems have been linked closely with the emergence and progress of electronic computers, user-friendly operation systems, application software and data-acquisition techniques. The ubiquitous availability of computers and the development of associated numerical methods have enabled hydrologists to carry out complex, repetitive calculations that use large quantities of data. Streamflow modelling has become an important element in the planning and management of water-supply and control systems and in providing river-forecast and warning services. The nature of modelling and the forced reliance on computer programming makes it impractical to include computational aids in this Guide. References are cited for further guidance on specific aspects of modelling, but no attempt is made to provide readily usable programs for the innumerable models that exist.

6.1 MATHEMATICAL DETERMINISTIC MODELS
[HOMS J04, J80, K22, K35, K55, L20]

There are many ways to classify mathematical models. For example, a model can be static or dynamic. A relationship between values of two variables, for example, between the river stage and discharge in a cross-section, in the same time instant can be interpreted as a static, or steady-state, model and described with the help of an algebraic equation. An example of a dynamic model is a quantitative relationship between the river flow in a cross-section of interest in a given time instant and a set of earlier values of rainfall over the basin terminated by this cross-section: rainfall-runoff models. Dynamic models are typically formulated in terms of differential equations, ordinary or partial. There are a number of dichotomy-type categorizations of dynamic models. For a discussion of these, see Singh (1988).

The category of dynamic hydrological models is very general and covers an entire spectrum of approaches. On one extreme are the purely empirical, black box techniques: those that make no attempt to model the internal structure but only match the input and output of the catchment system. A special category of black box models are artificial neural networks. On the other extreme are techniques involving complex systems of equations based on physical laws and theoretical concepts that govern hydrological processes: the so-called hydrodynamic models (see *Hydrological Model for Water-Resources System Design and Operation*, Operational Hydrology Reoort No. 34). Between these two extremes there are various conceptual models. These models represent a structure built of simple conceptual elements, such as, linear or non-linear reservoirs and channels that simulate, in an approximate way, processes occurring within the basin. Whether the models are black box, conceptual or hydrodynamic, they yield outputs without the possibility of evaluating associated probabilities of occurrence. For this reason, they are often referred to as deterministic models.

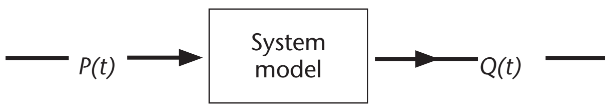
Lumped models have constant parameters, which do not change in space and are typically described by ordinary differential equations, while parameters of distributed models, whose physics is described by partial differential equations, may vary in space. Distributed and semi-distributed models have become common as they make use of the distributed data fields which are available from remote-sensing. Linear models are convenient to use because they may have closed-form solutions and obey the superposition principle, which non-linear models do not. Models can be stationary, in other words, time-invariant, if the input–output relationship and model parameters do not change with time. Otherwise, models are non-stationary: time-variant. Models can be continuous and hence described by differential equations and integrals, or discrete and described by difference equations and sums.

Purely empirical and black box relationships have proven and will continue to prove beneficial under certain circumstances, but they are subject to serious error when it becomes necessary to rely on them under conditions not previously experienced. Models that treat through theoretical concepts varied and interacting hydrological processes, namely, physically based models, are expected to be more trustworthy under such conditions, and experimentation with them holds great promise for science. Any attempt to classify deterministic models as hydrodynamic, conceptual or black box models admittedly forces a decision as to the degree of empiricism. The division of dynamic hydrological models is, to some extent, arbitrary in the sense that one person's empiricism may be another person's theory (Singh, 1988). Nevertheless, it has been deemed appropriate to follow such a division in the treatment of deterministic models.

6.1.1 **Black box models**

A river basin can be regarded as a dynamic system in which lumped parameters, which are invariant over the basin, transform the input factors, precipitation and snowmelt, into a hydrograph of outflow from the basin. The same is true for a river reach, except that the inflow at the upstream point or points must be treated as an additional input factor. Diagrammatically, such systems can be represented as shown in Figure II.6.1, where *P(t)* is the input and *Q(t)* is the output, both functions of time *t*. From the standpoint of dynamic systems theory, hydrological systems behave as linear systems if they satisfy the principle of superposition, namely, that the reaction of the system to a combination of inputs is equal to the sum of its responses to the separate inputs, and that the system parameters are independent of the system's inputs or outputs. The premise that the outflow hydrograph of a basin can be predicted from a sequence of precipitation and snowmelt only involves the assumption that the variability of other natural inputs, such as evapotranspiration, is small or negligible, or follows a known function of time.

Figure II.6.1. Black box system



The general expression for the relationship between input *P(t)* and output *Q(t)* of a lumped-parameter, linear dynamic system may be written as follows:

$$a_n(t)\frac{d^nQ}{dt^n} + a_{n-1}(t)\frac{d^{n-1}Q}{dt^{n-1}} + \ldots + a_1(t)\frac{dQ}{dt} + a_0(t)Q$$
$$= b_n(t)\frac{d^nP}{dt^n} + b_{n-1}(t)\frac{d^{n-1}P}{dt^{n-1}} + \ldots + b_1(t)\frac{dP}{dt} + b_0(t)P \qquad (6.1)$$

where the coefficients a_i and b_i are the parameters characterizing the properties of the system. The solution to equation 6.1 for zero initial conditions gives:

$$Q(t) = \int_0^t h(t,\tau)P(\tau)d\tau \qquad (6.2)$$

where the function $h(t,\tau)$ represents the response of the system at a time t to a single input impulse at time τ. There are numerous approaches to the representation of hydrological systems by formulations involving the influence function $h(t,\tau)$, also called impulse response. These can be expressed in terms of the coefficients a_i and b_i of equation 6.1. If the coefficients are constant in time, the system is time invariant and equation 6.2 becomes the Duhamel integral:

$$Q(t) = \int_0^t h(t-\tau)P(\tau)d\tau \tag{6.3}$$

It can be shown that the unit hydrograph concept and the routing techniques discussed in 6.3.2.2.5 and 6.3.4.3 are all examples of linear dynamic systems involving the principle of superposition.

Non-linear systems are those for which the superposition principle is not observed. In general, the response of a non-linear, lumped-parameter system to an input can be expressed either by an ordinary non-linear differential equation or by the integral equation:

$$\begin{aligned} Q(t) &= \int_0^t h(\tau)P(t-\tau)d\tau \\ &+ \int_0^t h(\tau_1,\tau_2)P(t-\tau_1)P(t-\tau_2)d\tau_1 d\tau_2 \\ &+ \int_0^t \dots \int_0^t h(\tau_1,\tau_2,\dots,\tau_n)P(t-\tau_1) \\ & P(t-\tau_2)\dots P(t-\tau_n)d\tau_1 d\tau_2 \dots d\tau_n \end{aligned} \tag{6.4}$$

where $h(\tau_1, \tau_2, \dots \tau_n)$ is a function expressing the time-invariant characteristics of the physical system. It is analogous to the influence function in equation 6.2. The first term on the right-hand side of equation 6.4 defines the linear properties of the system, while the second defines the quadratic properties; the third defines the cubic properties, and so on.

6.1.2 Artificial neural networks

A particular class of mathematical model is the artificial neural network, which is being increasingly used as an alternative way to solve a range of hydrological problems. The approach can be regarded as a modelling tool composed of a number of interconnected signal-processing units called artificial neurons. Artificial neural networks, which can capture and represent complex input–output relationships, resemble the parallel architecture of the human brain; however, the orders of magnitude are not as great. The idea behind the development of artificial neural networks was the desire to simulate the basic functions of the natural brain and develop an artificial system that could perform intelligent tasks similar to those performed by the brain. Artificial neural networks acquire knowledge through learning and store the acquired knowledge within inter-neuron connection or synaptic weights.

Artificial neural networks are a simple clustering of the primitive artificial neurons. Each neuron is connected to a number of its neighbours. This clustering occurs by creating layers, which are connected to one another. The connections determine whether it is possible for one unit to influence another. Some of the neurons in input and output layers interface with the real world: neurons in the input layer receive input from the external environment, while those in the output layer communicate the artificial neural network output to the external environment (Figure II.6.2). There are usually a number of hidden layers between the input and output layers.

When the input layer receives the input, its neurons produce an output which becomes an input to the next layer of the system. The process continues until the output layer fires the output to the external environment. An input–output function, or transfer function, should be specified for the artificial neural network units. For instance, the transfer function can follow a linear, threshold or sigmoid law. To construct a neural network that performs a specific task, the structure of the network and the scheme of connections between units must be chosen and the weights on the

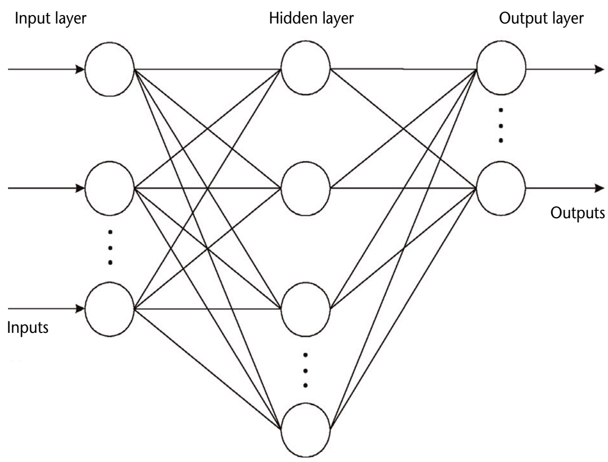


Figure II.6.2. Structure of an artificial neural network

connections specifying the strength of the connections must be set.

The learning ability of a neural network is determined by its architecture and the algorithm chosen for training. There are a variety of learning laws which are used in artificial neural networks. These laws are mathematical algorithms used to update the connection weights. Changing connection weights of an artificial neural network, known as training, causes the network to learn the solution to a problem. Gathering new knowledge is accomplished by adjusting connection weights in such a way that the overall network produces appropriate results. An artificial neural network developer must decide on the arrangement of neurons in various layers, on inter-layer and intra-layer connections, on the way a neuron receives input and produces output, and on the principle of the learning process. Determination of a number of hidden neurons in the network can be seen as an optimization task which is often done by means of trial and error. Excessive increase of the hidden number of neurons leads to an overfit, when generalization is difficult.

A range of artificial neural network architectures and training algorithms ranging from feed-forward artificial neural networks trained using back propagation to self-organizing maps for pattern discovery have been devised. Artificial neural networks are a quick and flexible approach which has been found suitable for hydrological modelling in a wide variety of circumstances. There are several applications of neural networks of interest to hydrology in areas such as rainfall-runoff modelling (see Minns and Hall, 1996), flow routing (Cigizoglu, 2003) and sediment transport (Tayfur, 2002). As neural networks are best at identifying patterns or trends in data, they are well suited to prediction or forecasting.

The principal advantage of neural networks lies in their ability to represent both linear and non-linear relationships and to learn these relationships directly from the data being modelled. Traditional linear models are simply inadequate when it comes to modelling data that contains non-linear characteristics, as is the case for most hydrological systems. At the start of the twenty-first century, a great deal of research is being conducted in neural networks and their application to solve a variety of problems worldwide. However, hydrological practices have not yet accommodated these methods on a routine basis. Well-established technologies are still preferred to novelties whose advantages are yet to be proved. Also, the black box nature of artificial neural networks has caused reluctance on the part of some hydrologists.

6.1.3 Conceptual models

The approaches discussed in the previous sections make use of only very general concepts of the transformation of input data into the outflow hydrograph, while more structural information about a system or a process may be available. Such an approach is inadequate for solving catchment modelling problems in which it is necessary to evaluate the effects of climate variability and change, changes in land use and other human activities. As a result, a modelling approach has been developed that involves structures based on various simplified concepts of the physical processes of flow formation. These are commonly referred to as conceptual models.

One of the most difficult aspects of applying conceptual models is the calibration of a chosen model to a particular catchment. Most of the parameters are determined by iterative processes, either automatic or manual, that use historical input-output data. Owing to data limitations, model imperfections and the interrelationships among the model parameters, a small increase in the number of parameters is likely to have a major, and negative, effect on the difficulty experienced in attempting calibration. It is necessary, therefore, that the number of parameters be compatible with the reliability of the input data and the required accuracy. In other words, modern concepts of theoretical merit must generally be simplified in favour of utility.

A wide variety of conceptual models are described in the literature (*Intercomparison of Conceptual Models Used in Operational Hydrological Forecasting* (WMO-No. 429)). Under the circumstances, it seems appropriate to limit the discussion to a brief description of three models, representing a reasonable cross-section of those suitable for treatment in this Guide. Several conceptual models are included in the Hydrological Operational Multipurpose System (HOMS) of WMO.

6.1.3.1 Sacramento model

The Sacramento model was developed by the staff of the National Weather Service River Forecast Center in Sacramento, California. This model embodies a complex moisture-accounting algorithm to derive volumes of several runoff components, while a rather simple and highly empirical method is used to convert these inputs to the outflow hydrograph. The soil mantle is treated in two parts, an upper zone and a lower zone, with each part having a capacity for tension water and free water. Tension water is water that is closely bound to the soil particles and depleted only by

evapotranspiration. Provision is made for free water to drain downward and horizontally. The storage capacities for tension water and free water in each zone are specified as model parameters. Water entering a zone is added to tension storage, as long as its capacity is not exceeded, and any excess is added to free water storage.

A portion of any precipitation is diverted immediately to the channel system as direct runoff. This is the portion that falls on the channel system and on impervious areas adjacent thereto. The extent of this area is time variant in the model. All rainfall and snowmelt, other than rainfall and snowmelt that are diverted to direct runoff, enter the upper zone. Free water in the upper zone is depleted either as interflow or percolation to the lower zone. If the rate of moisture supply to the upper zone is greater than the rate of depletion, the excess becomes surface runoff. Free water in the lower zone is divided between primary, slow drainage storage and secondary storage. Figure II.6.3 illustrates the principal features of the model.

Percolation from the upper to the lower zone is defined as:

$$PRATE = PBASE\left[1 + ZPERC * RDC^{REXP}\right]\frac{UZFWC}{UZFWM} \quad (6.5)$$

where *PRATE* is the percolation rate, and *PBASE* is the rate at which percolation would take place if the lower zone were full and if there were an unlimited supply of water available in the upper zone. It is numerically equal to the maximum lower zone outflow rate and is computed as the sum of the lower-zone primary and secondary free-water capacities, each multiplied by its depletion coefficient. *RDC* is the ratio of lower zone deficiency to capacity. That is, *RDC* is zero when the lower zone

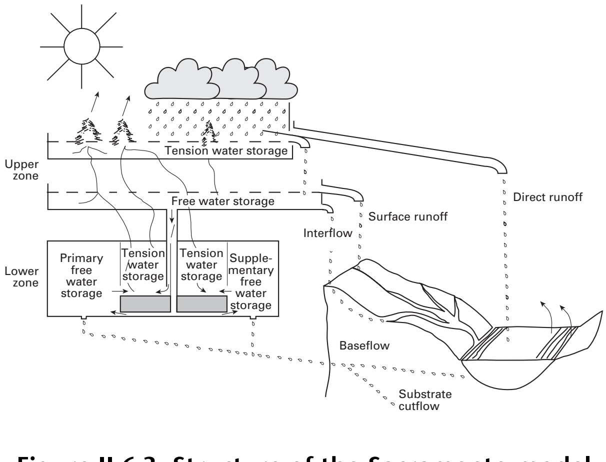


Figure II.6.3. Structure of the Sacramento model

is full and is in unity when it is empty. *ZPERC* is a model parameter that defines the range of percolation rates. Given an unlimited supply of upper zone free water, the rate will vary from *PBASE* (lower zone full) to *PBASE*(*1* + *ZPERC*) when the lower zone is empty. *REXP* is a model parameter that defines the shape of the curve between the minimum and maximum values described above. *UZFWC* is the upper zone freewater content. *UZFWM* is the upper-zone free capacity. The ratio, *UZFWC/UZFWM,* represents the upper zone driving force. With the upper zone empty, there will be no percolation. With it full, the rate will be governed by the deficiency in the lower zone.

This equation is the core of the model. It interacts with other model components in such a way that it controls the movement of water in all parts of the soil profile, both above and below the percolation interface, and, in turn, is controlled by the movement in all parts of the profile. Evapotranspiration rates are estimated from meteorological variables or from pan observations. Either day-by-day or long-term mean values can be used. The catchment potential is the product of the meteorological evapotranspiration and a multiplier that is a function of the calendar date, which reflects the state of the vegetation. The moisture accounting within the model extracts the evapotranspiration loss directly or indirectly from the contents in the various storage elements and/or from the channel system. The loss is distributed according to a hierarchy of priorities and is limited by the availability of moisture as well as by the computed demand.

The movement of moisture through the soil mantle is a continuous process. The rate of flow at any point varies with the rate of moisture supply and the contents of relevant storage elements. This process is simulated by a quasi-linear computation. A single time-step computation of the drainage and percolation process involves the implicit assumption that the movement of moisture during the time step is defined by the conditions existing at the beginning of the step. This approximation is acceptable only if the time step is relatively short. In the model, the length of the step is volume dependent. That is, the step is selected in such a way that no more than five millimetres of water may be involved in any single execution of the computational loop.

Five components of runoff are derived in the model. The three upper components – direct, surface and interflow – are summed and transformed by a unit hydrograph (see 6.3.2.2.5). The two components from the lower zone, and primary and secondary

base flow, are added directly to the outflow hydrograph derived from the other three components. Provision is also made for routing the resultant hydrograph with variable routing coefficients.

The Sacramento model is a HOMS component with the following identification code: J04.3.01.

6.1.3.2 Tank model

This model was developed at the National Research Institutes for Earth Science and Disaster Prevention in Tokyo, Japan (Sugawara and others, 1974). As the name implies, the soil mantle is simulated by a series of tanks arranged one above the other, as shown in Figure II.6.4 (a). All rainfall and snowmelt is assumed to enter the uppermost tank. Each tank has one outlet in the bottom and one or two on a side at some distance above the bottom. Water that leaves any tank through the bottom enters the next lower tank, except for the lowermost tank, in which case the downflow is a loss to the system. Water leaving any tank through a side outlet, known as sideflow, becomes input to the channel system. The number of tanks and the size and position of the outlets are the model parameters.

The configuration is a suitable representation of the rainfall-runoff process in humid regions, but a more complex arrangement is required for catchments in arid and semi-arid areas, as shown in Figure II.6.4 (b). If extended dry periods are typical, two or more series of tanks, as described above, are placed in a parallel arrangement. The downflows in each series are the same as for the simple tank model. Each tank in each series contributes sideflow to the corresponding tank of the next series, except that all sideflow from the last series feeds directly into the channel system. Additionally, provision is made for sideflow from the uppermost tank in all other series to feed directly into the channel system. Each series is considered to represent a zone of the catchment, the lowest corresponding to the zone nearest the channels. As hydrological conditions make their seasonal progression from wet to dry, the zone nearest the channels can continue to be relatively wet after the one furthest removed has become rather dry. The originators of the model do not claim that the representation of storage elements is entirely realistic, but rather that the configuration of tanks is an approximation somewhat resembling the finite-element method. Furthermore, the mathematical formulations defining the flow of water through the tanks resemble classical hydrological concepts.

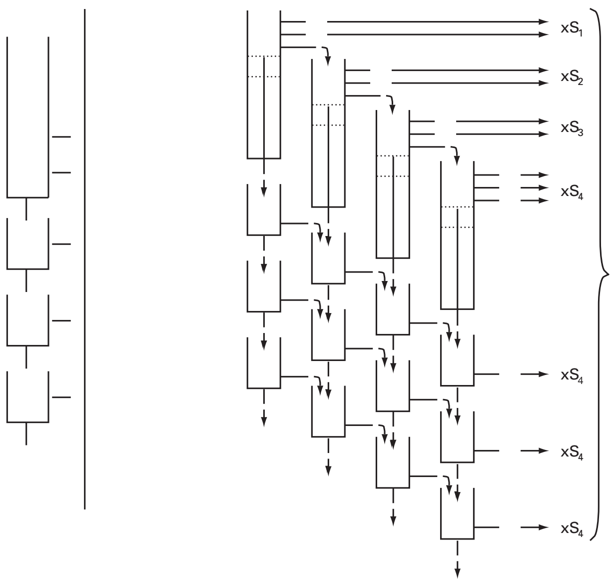


Figure II.6.4. Tank model

Two types of water are recognized in the model, confined water, namely, soil moisture, and free water that can drain both downward and horizontally. Provision is also made for free water to replenish soil moisture by capillarity action. The model computes evapotranspiration loss from the catchment based on measured or estimated daily evaporation, on the availability of water in storage, and on a hierarchy of priorities from the different storage elements.

The basic numerical calculation within a tank involves a withdrawal function defined by:

$$\frac{dx}{dt} = \alpha x \tag{6.6}$$

where x is the contents of the tank and t is time. The outflow in a finite unit of time, Δt, is therefore $[1 - e^{-\alpha\Delta t}]x$. The $[1 - e^{-\alpha\Delta t}]$ quantity is computed for each outlet, based on the value of α and the specified time interval.

The computation for each time interval proceeds in the following order:

(a) For the uppermost tank:
   (i) Extraction of evapotranspiration;
   (ii) Transfer of free water to soil moisture;
   (iii) Addition of rainfall and snowmelt;
   (iv) Calculation and extraction of channel system input (sideflow) and percolation (downflow) from free water contents;

(b) For a lower tank:
   (i) Extraction of evapotranspiration, depending on hierarchy of priorities;
   (ii) Transfer of free water to soil moisture;
   (iii) Addition of percolation water from tank immediately above;
   (iv) Calculation and extraction of channel system input (sideflow) and percolation (downflow) from free water contents.

The input to the channel system is the output from the moisture-accounting phase of the model. The outflow hydrograph is derived from the channel system input by routing, assuming that:

$$Q = KS^2 \qquad (6.7)$$

where Q is outflow, S is channel storage and the constant K is an additional parameter of the model. A limit of unity is imposed on dQ/dS to prevent the outflow from exceeding channel storage. An interesting feature of the tank model is that changes in the values of model parameters can actually change the structure of the model.

The tank model is a HOMS component with identification code J04.1.01.

6.1.3.3 HBV model

The HBV model, developed by Bergstrom (1992, 1995) at the Swedish Meteorological and Hydrological Institute, is a conceptual watershed model which converts precipitation, air temperature and potential evapotranspiration into snowmelt, if applicable, and streamflow or reservoir inflow. The model has been modified many times and exists in different versions in a number of countries.

The model describes the general water balance in the following way:

$$P - E - Q = \frac{d}{dt}\left[SP + SM + UZ + LZ + VL\right] \qquad (6.8)$$

where P is precipitation, E is evapotranspiration, Q is runoff, SP is snowpack, SM is soil moisture, UZ is upper groundwater zone, LZ is lower groundwater zone and VL is the volume of lakes.

The HBV model can be regarded as a semi-distributed model by dividing the catchment into sub-basins and using elevation zoning. The model contains subroutines for meteorological interpolation, snow accumulation and melt, evapotranspiration, soil moisture accounting, runoff generation and, finally, routing through rivers and lakes. For basins of considerable elevation range, a subdivision into elevation zones is made. Each elevation zone can be divided further into vegetation zones, such as forested and non-forested areas.

The standard snowmelt routine of the HBV model is a degree-day approach, based on air temperature. Melt is further distributed according to the elevation zoning and the temperature lapse rate and is modelled differently in forests and open areas. The snowpack is assumed to retain melt water as long as the amount does not exceed a certain water holding capacity of snow. When temperature decreases below the threshold temperature, this water refreezes gradually.

The soil moisture accounting of the HBV model is based on a modification of the tank approach in that it assumes a statistical distribution of storage capacities in a basin. This is the main control of runoff formation. Potential evapotranspiration is reduced to actual values along with a growing soil moisture deficit in the model and occurs from lakes only when there is no ice. Ice conditions are modelled with a simple weighting subroutine on air temperature, which results in a lag between air temperature and lake temperature.

The runoff generation routine is the response function which transforms excess water from the soil moisture zone to runoff. It also includes the effect of direct precipitation and evaporation on lakes, rivers and other wet areas. The function consists of one upper, non-linear, and one lower, linear, reservoir, producing the quick and slow, or base-flow, runoff components of the hydrograph. Lakes can also be modelled explicitly so that level pool routing is performed in lakes located at the outlet of a sub-basin. The division into submodels, defined by the outlets of major lakes, is thus of great importance for determining the dynamics of the generated runoff. River routing between sub-basins can be described by the Muskingum method, (see, for example, Shaw, 1994) or simple time lags.

A comprehensive re-evaluation of the model was carried out during the 1990s and resulted in the model version called HBV-96 (Lindström and others, 1997). The objectives were to improve the potential for accommodating spatially distributed data in the model, make the model more physically sound and improve the model performance. The model revision led to changes in the process descriptions, automatic calibration and optimal interpolation of precipitation and temperature, via

a geostatistical method. When combined, the modifications led to significant improvements in model performance. The option of higher resolution in space is also necessary for future integration of spatially distributed field data in the model. The improvements in model performance were due more to the changes in the processing of input data and the new calibration routine than to the changes in the process descriptions of the model.

Required input to the HBV model are precipitation (daily sums), air temperature (daily means) and estimates of potential evapotranspiration. The standard model is run with monthly data of long-term mean potential evapotranspiration, usually based on the Penman formula, adjusted for temperature anomalies (Lindström and Bergström, 1992). As an alternative, daily values can be calculated as being proportional to air temperature, but with monthly coefficients of proportionality. Later versions of the HBV model can be run with data of higher resolution in time, that is, hourly data.

Although the automatic calibration routine is not a part of the model itself, it is an essential component in practice. The automatic calibration method for the HBV model developed by Lindström (1997) has options for use of different criteria for different parameters or for use of combined criteria. This process generally requires three to five years of simultaneous streamflow and meteorological records. If no streamflow records are available, the parameters can, in some cases, be estimated from known basin characteristics.

The area of applicability of HBV is broad and embraces spillway design (Bergström and others, 1992; Lindström and Harlin, 1992), water resources evaluation, nutrient load estimates (WMO, 2003), and climate change studies (Bergström and others, 2001). A recent trend is the use of the model for nation-wide hydrological mapping, as in Norway (Beldring and others, 2003) and Sweden (SNA, 1995). The HBV model is a HOMS component with identification code J04.2.02. For further information, see: http://www.smhi.se/sgn0106/if/hydrologi/hbv.htm.

6.1.4 **Distributed models**

The field of mathematical modelling in hydrology has been traditionally dominated by lumped models with constant parameters, representing a whole drainage basin. However, several semi-distributed and distributed models have been developed more recently. Their formulation aims at following the hydrological processes more closely and thus may incorporate several meteorological variables and watershed parameters. Their product can be, for example, a series of synthetic streamflow data, water quality characteristics and rates of groundwater recharge. The basic input is a dataseries of rainfall data; however, provisions may be made for factors such as snowfall, temperature, radiation and potential evapotranspiration. Models for urban catchments may contain a description of their drainage network. Models for rural catchments may contain unit hydrographs, time-area curves or routing subroutines.

However, the distributed, physically based models are still being used at a fraction of their potential (Refsgaard and Abbott, 1996). There are several reasons for this. Distributed models require a large amount of data that often do not exist or are not available. Operational remote-sensing is still not a common practice, except for snow cover and land use/vegetation.

Numerous parameters of a physically based distributed model cannot be measured in the field and calibration of such a model is a difficult optimization task. Moreover, more complex and justified descriptions are rarely implemented in distributed models because they would require more parameters, which need to be identified. This simplification may undermine the rigour of the physical basis.

As noted by Beven (1996), physically based distributed models use small-scale equations with an assumption that the change of scales can be accommodated by the use of effective parameter values. However, physically based equations of small scale do not scale up easily in a heterogeneous system. Beven (1996) saw a possible solution in an approach that recognizes the limitations of the modelling process, such as within an uncertainty framework. Scale-dependent parameters could be based on a statistical model of heterogeneity. In general, efficient aggregate parameterization is not a trivial matter.

Distributed models provide a basis for full use of distributed information relevant to the physical processes in the catchment. The European Hydrological System (DHI, 1985) is an example of a distributed hydrodynamically based model and is illustrated in Figure II.6.5. The European Hydrological System is a model with distributed parameters that has been developed from partial differential equations describing the physical processes in the basin: interception, evapotranspiration, overland and channel flow, movement of water

through unsaturated and saturated zones, and snowmelt.

The interception process is represented by a variant of the Rutter model that gives the rate of change in the amount of water stored on the canopy:

$$\frac{\partial c}{\partial t} = Q - K e^{b} (C - S) \tag{6.9}$$

$$\text{where: } Q = \begin{cases} P_1 P_2 (P - E_p C / S) & \textit{when } C < S \\ P_1 P_2 (P - E_p) & \textit{when } C \geq S \end{cases}$$

C is the actual depth of water on the canopy, S is the canopy storage capacity, P is the rainfall rate, P_1 is the proportion of ground in plan view hidden by vegetation, P_2 is the ratio of total leaf area to area of ground covered by vegetation, E_p is the potential evaporation rate, K and b are drainage parameters and t is time.

For the prediction of actual evapotranspiration rates, the Penman–Monteith equation is used:

$$E_a = \frac{\Delta R_n \dfrac{\varphi C_p v_e}{r_a}}{\lambda \left[\Delta + \gamma \ (17 \gamma_s / r_a \right]} \tag{6.10}$$

where φ is the density of air, λ is the latent heat of vaporization of water, E_a is the actual evapotranspiration rate, R_n is the net radiation minus the energy flux into the ground, Δ_x is the slope of the specific humidity/temperature curve, C_p is the specific heat of air at constant air pressure, v_e is vapour pressure deficit of the air, r_a is the aerodynamic resistance to water vapour transport, γ_s is the canopy resistance to water transport, and γ is the psychometric constant.

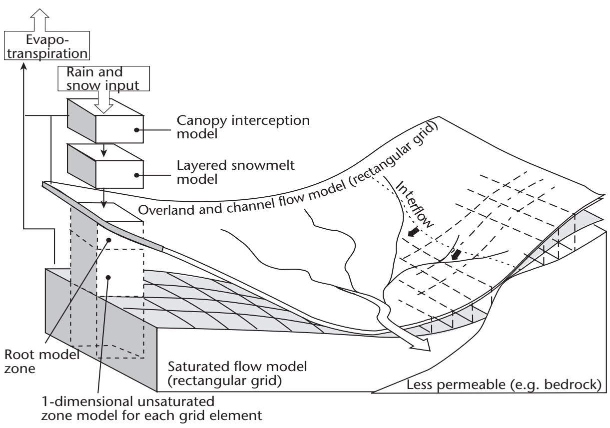


Figure II.6.5. Structure of the European Hydrological System

The interception process is modelled as an interception storage, which must be filled before stem flow to the ground surface takes place. The size of the interception storage capacity, I_{max}, depends on the vegetation type and its stage of development, which is characterized by the leaf area index, *LAI*. Thus:

$$I_{max} = C_{int} \times LAI \tag{6.11}$$

where C_{int} is an interception coefficient which defines the interception storage capacity of the vegetation. A typical value is about 0.05 mm, but a more exact value may be determined through calibration. The area of leaves above a unit area of the ground surface is defined by the leaf area index. Generalized time-varying functions of the leaf area index for different crops have been established. Thus, when employing the modelling tool MIKE SHE, the user must specify the temporal variation of the leaf area index for each crop type during the growing seasons to be simulated. Climatic conditions differ from year to year and may require a shift of the leaf area index curves in time but will generally not change the shape of the curve. Typically, the leaf area index varies between 0 and 7. Evaporation from the canopy storage is equal to the potential evapotranspiration, if sufficient water has been intercepted on the leaves, that is:

$$E_{can} = min\ I_{max} E_p \Delta t \tag{6.12}$$

where E_{can} is the canopy evaporation, E_p is the potential evapotranspiration rate and Δt is the time step length for the simulation.

Water accumulated on the soil surface responds to gravity by flowing downgradient over the land surface to the channel system, where it subsequently discharges through the stream channels to the catchment outlet. Both phenomena are described by equations of unsteady free-surface flow, based on physical principles of conservation of mass and momentum (DHI, 1985).

In the most comprehensive mode, the flow in the unsaturated zone can be computed by using the Richards equation:

$$C = \frac{\partial \Psi}{\partial t} = \frac{\partial}{\partial Z} \left(K \frac{\partial \Psi}{\partial Z} \right) + \frac{\partial K}{\partial Z} + S \tag{6.13}$$

where Ψ is the pressure head, t is the time variable, Z is the vertical coordinate (positive upwards), $C = \partial \Theta / \partial \Psi$ is the soil-water capacity, Θ is the volumetric moisture content, K is hydraulic conductivity and S is a source/sink term.

The infiltration rate into the soil is determined by the upper boundary condition, which may shift from flux-controlled conditions to soil-controlled or saturated conditions and vice versa. The lower boundary is usually the phreatic-surface level. The governing equation describing the flow in the saturated zone is the non-linear Boussinesq equation:

$$S = \frac{\partial h}{\partial t} = \frac{\partial}{\partial x}\left(K_x H \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y H \frac{\partial h}{\partial y}\right) + R \quad (6.14)$$

where S is the specific yield; h is the phreatic surface level; K_x, K_y are the saturated hydraulic conductivities in the x and y directions, respectively; H is the saturated thickness; t is the time variable; x, y are the horizontal space coordinates and R is an instantaneous recharge/discharge term.

Equation 6.14 is solved by approximating it to a set of finite difference equations, that is, by applying Darcy's law in combination with the mass balance equation for each computational node. Considering a node i inside the model area, the total inflow R from neighbouring nodes and sources/sinks between time n and time $n+1$ is expressed as follows:

$$R = \sum q_z^{n+1} + \sum q_x^{n+1} + RH_i \Delta x^2 \quad (6.15)$$

where the first term on the right-hand side is the volumetric flow in the vertical direction, the second term is the volumetric flow in horizontal directions, R is the volumetric flow rate per unit volume from all sources and sinks, Δx is the spatial resolution in the horizontal direction and H_i is either the saturated depth for unconfined layers or the layer thickness for confined layers.

The snowmelt component of SHE represents an attempt to model both energy and mass flux within a snowpack by taking into account changes in the structure of the pack. Two semi-empirical equations are used to complete the set of relationships required to define temperature and water-content distributions. Empirical equations are also used to define the hydraulic and thermal properties of the snow in terms of the structure, water content and temperature.

There have been several products linked to SHE, including MIKE SHE, SHETRAN, or SHESED developed recently. Basic description of processes from the original SHE remains in MIKE SHE. This latter package (Storm and Refsgaard, 1996) extended in comparison to SHE, has been used in a number of practical applications, including flow simulation, solute transport, applications in irrigation and salinity planning, and management models.

6.1.5 **Parameter evaluation**

General methods of parameter evaluation or identification sometimes referred to as model calibration, have been developed for a wide range of dynamic systems. Experience has shown that the success of such methods depends on the availability of adequate information concerning the system characteristics and the form of the influence function, or impulse response. There are two basic approaches to calibration.

In the first approach, the mathematical model is combined with the data to solve for the unknown coefficients, the system parameters. Such a task belongs to the category of ill-posed mathematical inverse problems and is usually difficult to solve. In the linear case, matrix inversion may be needed. The solutions can be very sensitive to inaccuracy in the data. They tend to be unstable and multiple solutions may exist. The optimum found by the optimization software can be a local, rather than a global optimum.

The second approach involves experimentation with various combinations of parameter values in an effort to minimize or maximize an adopted criterion of optimization. A number of strategies have been developed by applied mathematicians with a view to minimizing the number of calculations required in the optimization of parameter values. Among the strategies used in hydrology are the gradient and non-gradient methods. The adequacy of the solution can be highly dependent on the criterion used in the analysis. A number of criteria have been developed and introduced through WMO projects (WMO, 1986, 1987, 1991*a*). These can be recommended for general use.

To determine the parameters of complex, conceptual hydrological models which have several components, the following principles are recommended:

(a) Separate testing of the model components using all available experimental and scientific information. It is well known that the global determination of all parameters of a model through optimization may result in unrealistic values of the parameters, some even falling outside their physical range. This is the case when certain model components contain systematic errors that are subsequently compensated within the model. In order to avoid such situations, it is

recommended that the parameters of complex conceptual models be determined separately for each of the basic components and not globally;

(b) Data from a minimum three-year time interval should be used for the calibration of models, and another time interval of similar length should be used for verification. The calibration and verification intervals in this split-sample approach should be selected so that they represent different conditions favouring runoff formation, for example: floods generated by rainfall, floods resulting from snowmelt processes and low flows;

(c) In the case of basins with a hydrological regime under anthropogenic influences, it is recommended that the model be calibrated for the natural runoff regime. The values of certain parameters may be modified subsequently to account for human influences. The validation of the model parameters should be done for a representative period that is not influenced by human activities.

The parameters of hydrodynamic models represent basin characteristics, such as roughness of the slopes and the river bed, soil hydraulic conductivity and soil porosity. In principle, all of these parameters are physically based and determined by field measurements and not through optimization. However, this is not always possible in practice.

6.1.6 Selection of models

In addition to software packages developed in Europe and North America, several products from other countries are being increasingly used in the international context. For example, two models developed in South Africa have gained international recognition. The ACRU (Agricultural Catchments Research Unit) agrohydrological modelling system developed by Schulze at the University of Natal since the early 1970s is a multipurpose integrated physical conceptual model simulating streamflow, sediment and crop yields. The Pitman rainfall-runoff model for monthly time steps has been widely used in southern Africa for broad strategic water resource planning purposes (see Hughes and Metzler, 1998). Recently, Hughes (2004*a*) extended the Pitman model by adding two new components, recharge and groundwater discharge, thus responding to the urgent need in practice for an integrated surface water and groundwater modelling tool which can be applied at various basin scales in southern African conditions.

The choice of models is not restricted to the models described above. Many models produced by research institutions and commercial software companies are available. It is often difficult to ascertain the relative advantages and disadvantages of models proposed for operational use. The selection of a model suitable for a specific hydrological situation has implications in water resources planning, development and management; in hydrological forecasting activities and in setting directions of further research in modelling. Some of the factors and criteria involved in the selection of a model include the following:

(a) The general modelling objective: hydrological forecasting, assessing human influences on the natural hydrological regime or climate change impact assessment;
(b) The type of system to be modelled: small watershed, aquifer, river reach, reservoir or large catchment;
(c) The hydrological element to be modelled: floods, daily average discharges, monthly average discharges, groundwater levels, water quality and so forth;
(d) The climatic and physiographical characteristics of the watershed;
(e) Data availability with regard to type, length and quality of data versus data requirements for model calibration and operation;
(f) Model simplicity, as far as hydrological complexity and ease of application are concerned;
(g) The possible need for transposing model parameters from smaller catchments to larger catchments;
(h) The ability of the model to be updated conveniently on the basis of current hydrometeorological conditions.

Useful information and guidance on the selection and application of conceptual models in various hydrological situations can be found in documentation of several WMO projects carried out since the 1970s, such as the following:

(a) Intercomparison of conceptual models used in operational hydrological forecasting (WMO, 1987);
(b) Intercomparison of models of snowmelt runoff (WMO, 1986);
(c) Simulated real-time intercomparison of hydrological models (WMO, 1991*a*).

Many hydrological software packages have been developed by scientific research institutes and commercial companies for PCs and work stations using MS Windows, UNIX and LINUX platforms. Many models are equipped with a geographical information system interface.

Hydrological models within HOMS are grouped in a number of sections. Section J, Hydrological Forecasting Models, includes models whose main purpose is the operational forecasting of various hydrological elements. Subsection J04, Forecasting Streamflow from Hydrometeorological Data, includes the three models, Sacramento, tank and HBV, introduced in 6.1.3.1 to 6.1.3.3.

At the time of writing, further components in this subsection of HOMS include the following: J04.1.04, Snowmelt-runoff model (SRM); J04.1.05, Inflow–storage–outflow (ISO) function models; J04.2.01, Conceptual watershed model for flood forecasting; J04.3.03, Snow accumulation and ablation model (NWSRFS-SNOW-17); and J04.3.07, Synthetized constrained linear system (SCLS).

Subsection J15, Combined Streamflow Forecasting and Routing Models, includes components J15.2.01, Streamflow synthesis and reservoir regulation (SSARR) model, and J15.3.01, Manual calibration program (NWSRFS-MCP3).

Further models are grouped under section K, Hydrological Analysis for the Planning and Design of Engineering Structures and Water Resource Systems), for example, K15, Site-Specific Flood Studies, and K15.3.02, Dam-break flood model (DAMBRK). Subsection K22, Rainfall-Runoff Simulation Models, contains K22.2.02, Flood hydrograph package (HEC-1); K22.2.10, Hydrological rainfall runoff model (HYRROM); K22.2.11, Unit hydrographs and component flows from rainfall, evaporation and streamflow data (PC IHACRES); K22.2.12, Non-linear rainfall-runoff model (URBS) and K22.3.01, Urban rainfall-runoff model (SWMM). Subsection K35, Streamflow Simulation and Routing, includes the following components: K35.1.05, Numerical solutions of the non-linear Muskingum method; K35.2.09, Interior flood hydrology (HEC-IFH); K35.3.06, River analysis system (HEC-RAS); K35.2.06, Water-surface profile computation model (WSPRO); K35.3.13, Branch-network dynamic flow model (BRANCH); and K35.3.14, Flow model for a one-dimensional system of open channels based on the diffusion analogy (DAFLOW). Subsection K 55, Water Quality Studies, includes the following components: K55.2.04, Transport model for a one-dimensional system of open channels (BLTM); K55.2.06, Modelling faecal coliform concentrations in streams; K55.3.04, Mathematical model for two-dimensional salinity distribution in estuaries; and K55.3.07, PC-QUASAR – Quality simulation along rivers.

Section L, Groundwater, includes subsection L20, Aquifer Simulation Models, with the following components: L20.2.04, Modular finite-difference groundwater flow model (MODFLOW); L20.3.05, A model for unsaturated flow above a shallow water table (MUST); L20.3.13, Complete program package for modelling groundwater flow (TRIWACO); L20.3.07, Pathlines and travel times based on analytical solutions (AQ-AS); L20.3.10, Groundwater head drawdowns based on analytical solutions (AQ-AP); L20.3.11, Aquifer simulation model; L20.3.12 , SGMP – Simulation of watertable behavior in groundwater systems; and L20.3.14, MicroFEM – Finite-element multiple-aquifer steady-state and transient groundwater flow modelling.

6.2 TIME SERIES AND SPATIAL ANALYSIS

Many hydrological data consist of time series of observations of a hydrological variable in one point in space. Studying a single time series of hydrological data allows the identification of the temporal correlation structure of this variable in one point in space. If more than one variable is being observed at the station, cross-correlations between time series of several variables in the same spatial point can be studied. By considering time series of the same variable at a number of points in space, a spatial-temporal field of this hydrological variable can be explored and a cross-correlation between the time series of the same variable in different spatial points can be examined. This makes it possible to interpret the temporal and/or spatial-temporal structure of the hydrological processes and to use this in synthetic streamflow generation and extending data, such as for filling in gaps in data and extrapolation.

Hydrological time series can be continuous in that they are derived from a continuously recording device, discrete because they are sampled at discrete time instants at regular or irregular time intervals, or quantized, if each value of the time series is an integral of a variable over a defined time interval. Continuous time series can be analysed in the temporal domain or in the operational domain, for example, via Fourier or Laplace integral transforms, which can be convenient in specific cases.

When studying hydrological time series, it is important to use appropriate time intervals. Data may be hourly, daily, monthly or annual, but in a particular application it may be necessary to use either the time interval dictated by the data acquisition, or a

longer period, requiring aggregation, or a shorter period, requiring disaggregation. This has effects on the characteristics of the series. Series of hourly streamflow are very likely to contain highly correlated values, while the correlation coefficient may fall to zero in a series of annual values.

There is extensive literature related to time series analysis, including the monumental monograph by Box and Jenkins (1970). Hydrological applications of time series analysis can be studied in Salas (1992). Elements of time-series analysis are widely included in general-purpose statistical software packages. The present section briefly describes practical problems in the field, dealing with stochastic simulation and change detection in hydrological records.

6.2.1 Stochastic simulation of hydrological times series

Stochastic models are black box models, the parameters of which are estimated from the statistical properties of the observed times series. Stochastic methods were first introduced into hydrology in connection with the design of storage reservoirs. Annual or monthly flow volumes provide adequate detail for such purposes, but the capacity of the reservoir must reflect the probability of occurrence of critical sequences of flow that can best be evaluated from a set of flow sequences. Each must span a period of many years and should be indistinguishable from the historic record in so far as its relevant statistical characteristics are concerned. The statistical properties of the historical record that are to be preserved are of primary concern in the selection of an appropriate stochastic model. Modelling is much more difficult when it becomes necessary to generate simultaneous flow sequences for two or more reservoir sites in a basin because of the requirement that intercorrelations be preserved. Stochastic modelling has also been used in the establishment of confidence limits of real-time flow forecasts. Such applications are not given further treatment here. A discussion of the design and operation of storage reservoirs is provided in 4.2.

6.2.1.1 Markovian lag-1 models

Many models for simulating monthly, seasonal or annual flow volumes assume a first-order Markov structure which assumes that the flow in any period is determined by the flow in the preceding period, plus a random impulse. One such model for annual flows can be expressed as:

$$Q_i = \bar{Q}_j + \rho_j \frac{\sigma_j}{\sigma_{j-1}}(Q_{i-1} - \bar{Q}_{j-1}) + \varepsilon_i \sigma_j \sqrt{1 - \rho_j^2} \qquad (6.16)$$

in which Q_i is the flow of the i^{th} member of the series numbering consecutively from 1 regardless of month or year, j is the month in which the *ith* member of the series falls, $\bar{Q}_j$ is the mean flow for the jth month, σ_j is the standard deviation for the *jth* month, ρ_j is the serial correlation between $\bar{Q}_j$ and Q_{j-1} and ε_i is a random variate from an appropriate distribution, with mean zero, unit variance and serial independance.

Equation 6.16 is also suitable for seasonal flows (j = 1, 2, 3, 4) and annual flows ($j = 1$). In the latter case it becomes:

$$Q_i = \bar{Q}_j + \rho(Q_{i-1} - \bar{Q}_{j-1}) + \varepsilon_i \sigma \sqrt{1 - \rho^2} \qquad (6.17)$$

Values of $\bar{Q}$ σ and ρ, derived from the historical record, are assumed to be applicable for the purposes to be served, and an initial value of Q_{i-1} need only be selected to simulate a series of any length. Monte Carlo techniques are generally used with sequential values of the random variate derived by computer. In principle, the development and application of the models depicted in equation 6.16 are relatively straightforward and simple. Nevertheless, several questions requiring careful consideration and decisions may be critical to the particular problem under study:

(a) What is the distribution of the random variate?
(b) Should the variance be corrected for serial correlation, if present?
(c) How accurate is the calculated value of the serial correlation?

6.2.1.2 Autoregressive moving average models

An important extension of the univariate stochastic models is represented by the group developed by Box and Jenkins (Box and Jenkins, 1970; Hipel and others, 1977): the autoregressive moving average models (ARMA).

There are three types: autoregressive (AR), moving average (MA) and mixed (ARMA) models. The most general type (ARMA), of order p and q, and the moving average (MA), of order q, are, respectively:

$$x_i = \phi_1 x_{i-1} + \phi_2 x_{i-2} + \ldots + \phi_p x_{i-p} + \varepsilon_i - \theta_1 \varepsilon_{i-1} - \ldots - \theta_q \varepsilon_{i-q} \qquad (6.18)$$

$$x_i = \varepsilon_1 - \theta_i \varepsilon_{i-1} - \ldots - \theta_q \varepsilon_{i-q} \qquad (6.19)$$

where x_i is the deviation of the i^{th} observation from the series mean, ϕ_1 and θ_1 are parameters to be estimated, and ε_i is a random variate as defined above (see 6.2.2.1).

A systematic approach has been developed for fitting ARMA models (Box and Jenkins, 1970):
(a) Identification: The correlogram of the series under study is compared with the autocorrelation functions of various ARMA models as a basis for selecting the appropriate type and order;
(b) Estimation: The parameters of the model are estimated (Salas, 1992) by using the method of moments, the method of maximum likelihood or the method of least squares, where estimates minimizing the sum of squared residuals are selected;
(c) Diagnostic checking: The randomness of the residuals is checked to verify the adequacy of the selected model.

Autoregressive moving average models are used to generate synthetic flow sequences by Monte Carlo techniques in the manner previously described. It is important to bear in mind that methods of stochastic generation should be used with caution and with critical consideration of the characteristics of the record that are important for the water resource project under study.

6.2.1.3 Fractional Gaussian noise and broken-line process models

Hurst discovered (Hurst, 1951) that very long geophysical records displaying characteristics at odds with stationary Markovian processes led to the development of two stochastic models that can accommodate long-term persistence or low-frequency elements. The first of these, the fractional Gaussian-noise (FGN) model (Mandelbrot and Wallis, 1968) is a self-similar, random process characterized by a spectral density function that emphasizes very low frequencies typifying the Hurst phenomenon. It also has been shown that a long-memory model of the broken-line process will preserve the Hurst phenomenon (Rodriguez-Iturbe and others, 1972; Mejia and others, 1972).

The findings of Hurst do not necessarily indicate very long-term persistence and, moreover, some versions of ARMA models are capable of simulating substantial low-frequency effects. The non-stationarity of the process's mean value could also result in the characteristics that Hurst found when analysing long records, whether these be the result of climatic change, anthropogenic factors or simply non-homogeneity of the data series.

6.2.2 Change detection in hydrological records

6.2.2.1 Introduction

Detection of changes in long time series of hydrological data is an issue of considerable scientific and practical importance. It is fundamental for planning of future water resources and flood protection. If changes are occurring within hydrological systems, existing procedures for designing structures such as reservoirs, dams and dykes will have to be revised; otherwise, systems will be over- or under-designed and will either not serve their purpose adequately or will be more costly than necessary.

Activities undertaken within the World Climate Programme – Water (WMO, 1988) have led to the establishment of general recommendations on methodology for use in detection of change in hydrological data, presented by Cavadias (WMO, 1992) and Kundzewicz and Robson (WMO, 2000, 2004). The present section is based on the latter two references, which may be consulted for more detailed recommendations regarding different components of the process of testing for changes.

6.2.2.2 Basics of statistical testing for change detection

Change in a time series can occur in numerous ways: gradually (a trend), abruptly (a step change) or in a more complex form. It may affect the mean, median, variance, autocorrelation or other aspects of the data.

In order to carry out a statistical test, it is necessary to define the null and alternative hypotheses which describe what the test is investigating. For example, to test for trend in the mean of a series, the null hypothesis would be that there is no change in the mean of a series, and the alternative hypothesis would be that the mean is either increasing or decreasing over time. To perform a test, it is necessary to begin by assuming that the null hypothesis is true. The next step is to check whether the observed data are consistent with this hypothesis. If not, the null hypothesis is rejected.

To compare between the null and alternative hypotheses, a test statistic is selected and its significance is evaluated, based on the available evidence. The test statistic is simply a numerical value that is

calculated from the data series that is being tested. A good test statistic should highlight the difference between the two hypotheses. A simple example of a test statistic is the linear regression gradient, which can be used to test for a trend in the mean. If there is no trend (null hypothesis), the regression gradient should have a value near to zero. If there is a large trend in the mean (alternative hypothesis), the value of the regression gradient would be very different from zero: positive for an increasing trend and negative for a decreasing trend.

The significance level measures whether the test statistic differs significantly from the range of values that would typically occur under the null hypothesis. It is the probability that a test erroneously detects trend when none is present; this is referred to as a type I error. A type II error occurs when the null hypothesis is accepted – no trend is present – when in fact the alternative hypothesis is true (a trend exists). The power of a test is the probability of correctly detecting a trend when one is present; powerful tests that have a low type II error probability are preferred.

In carrying out a statistical test it is always necessary to consider assumptions. Standard tests require some or all of the following assumptions: a specified form of distribution, for example, assuming that the data are normally distributed; constancy of the distribution in that all data points have an identical distribution so that there are no seasonal variations or any other cycles in the data; and independence. This last assumption is violated if there is either autocorrelation, namely, correlation from one time value to the next. This is also referred to as serial correlation or temporal correlation or, in the case of a multi-site study, spatial correlation, in particular, correlation between sites.

If the assumptions made in a statistical test are not fulfilled by the data, then test results can be meaningless, in the sense that estimates of the significance level would be grossly incorrect. Hydrological data are often clearly non-normal; this means that tests that assume an underlying normal distribution will be inaccurate. Hydrological data may also show autocorrelation and/or spatial correlation; therefore, data values are not independent. This can have a negative impact on the ability to detect trend in a time series (Yue and others, 2003). The data may also display seasonality, which violates assumptions of constancy of distribution. The power of tests can depend on the sample size, the variability of the time series, the magnitude of the characteristic that is being tested, such as a trend, and the distribution and skewness of the time series. Results of the power of the Mann–Kendall and Spearman's rho tests are provided by Yue and others (2002) and Yue and Pilon (2004).

The main stages in statistical testing are as follows:

(a) Decide what type of series/variable to test, depending on the issues of interest, for example, monthly averages, annual maxima or deseasonalized data;
(b) Decide what types of change are of interest (trend/step change);
(c) Check out data assumptions, for instance by using exploratory data analysis;
(d) Select one or more tests/test statistics that are appropriate for each type of change; more than one is good practice;
(e) Select a suitable method for evaluating significance levels;
(f) Evaluate significance levels;
(g) Investigate and interpret results.

The process of selecting a statistical test can be considered to be composed of two parts: selecting the test statistic and selecting a method for determining the significance level of the test statistic. By viewing the process in manner, it becomes possible distinguish between how to select a test statistic and how to evaluate the significance level.

6.2.2.3 Distribution-free testing

There are many ways of testing for trend or other changes in hydrological data. In a particular group of methods, referred to as distribution-free methods, there is no need for assumptions as to the form of distribution from which the data were derived, for example, it is unnecessary to assume data are normally distributed. The following approaches are distribution-free:

(a) Rank-based tests: These tests use the ranks of the data values but not the actual data values. A data point has rank r if it is the rth largest value in a dataset. Most rank-based tests assume that data are independent and identically distributed. Rank-based tests have the advantage that they are robust and usually simple to use. They are generally less powerful than a parametric approach.
(b) Tests using a normal-scores transformation: Many tests for change rely on the assumption of normality. They are generally not suitable for direct use with hydrological data, which are typically far from being normally distributed. However, such tests can be used if the data are first transformed. The normal scores transformation results in a dataset that has a normal distribution. It is similar to using the ranks of

a data series, but instead of replacing the data value by its rank, r, the data value is replaced by the typical value that the rth largest value from a sample of normal data would have (the rth normal score). The advantages of using normal scores are that the original data need not follow a normal distribution, and the test is relatively robust to extreme values. The disadvantage is that statistics measuring change, such as the regression gradient, cannot be easily interpreted. Normal-scores tests are likely to be slightly more powerful than equivalent rank-based tests.

(c) Tests using resampling approaches: Resampling methods, introduced below, are methods that use the data to determine the significance of a test statistic.

6.2.2.4 Introducing resampling methods

Resampling methods, permutation testing and the bootstrap method are a robust set of techniques for estimating the significance level of a test statistic. They are flexible and can be adapted to a wide range of types of data, including autocorrelated or seasonal data, and are relatively powerful. Resampling methods are very useful for testing hydrological data because they require relatively few assumptions to be made about the data, yet they are also powerful tests. They provide a flexible methodology that allows significance levels to be estimated for any sensible choice of test statistic. They enable traditional statistical tests to be adapted for application to hydrological series by using a robust method to determine significance.

The basic idea behind re-sampling methods is very straightforward. Consider testing a series for trend: a possible test is the regression gradient. If there is no trend in the data (the null hypothesis) then the order of the data values should make little difference. Thus shuffling, or permuting, the elements of the data series should not change the gradient significantly. Under a permutation approach the data are shuffled very many times. The test statistic is recalculated after each shuffle or permutation. After many permutations, the original test statistic is compared with the generated test statistic values. If the original test statistic differs substantially from most of the generated values, this suggests that the ordering of the data affects the gradient and that there was trend. If the original test statistic lies somewhere in the middle of the generated values, then it seems reasonable that the null hypothesis was correct in that the order of the values does not matter; hence there is no evidence of trend. In other words, if an observer or, in this case, the statistical test can distinguish between the original data and the resampled or permuted data, the observed data are considered not to satisfy the null hypothesis.

The bootstrap and permutation methods are two different approaches to resampling the data. In permutation methods, sampling with no replacement, the data are reordered, that is each of the data points in the original data series appears only once in each resampled or generated data series. In bootstrap methods, the original data series is sampled with replacement to give a new series with the same number of values as the original data. The series generated with this method may contain more than one of some values from the original series and none of other values. In both cases, the generated series has the same distribution as the empirical, observed distribution of the data. In general, bootstrap methods are more flexible than permutation methods and can be used in a wider range of circumstances.

The simplest resampling strategy is to permute or bootstrap individual data points, as described above. This technique is applicable only when it can be assumed that the data are independent and non-seasonal. If data show autocorrelation, or additional structure such as seasonality, the series generated by resampling should replicate this structure. A straightforward means of achieving this is to permute or bootstrap the data in blocks. For example, for a 40-year series of monthly values, it would be sensible to treat the data as consisting of 40 blocks of one year. Each year's worth of data is left intact and is moved around together as a block, thus maintaining the seasonal and temporal dependencies within each year. The 40 blocks are then reordered many times. In this way, the resampled series will preserve the original seasonality. Similarly, blocks can be forced to replicate the autocorrelation in the data. It is important that the size of the blocks should be sensibly selected.

Many distribution-free tests, such as the rank-based tests, depend on assumptions of independence. If this assumption does not hold, as is common for hydrological data, the recommended approach is to extract the test statistics from these tests and to evaluate significance using block-bootstrap and block-permutation methods, rather than using classical formulae for significance, which may lead to gross errors. Such methods can be useful when there is spatial dependency in a set of multi-site data that is to be tested as a group. In this case, the usual choice of blocks would be to group data across all sites that occurred in the same time interval (for example, Robson and others, 1998).

6.2.2.5 Commonly used tests and test statistics

Table II.6.1 presents a summary of standard parametric and non-parametric tests for change detection, their essential properties and necessary assumptions. The tests are described in their standard form, namely, in a non-resampling framework. Each of these tests can easily be adapted to be a resampling test. For this, the test statistic for a test is calculated, but the significance level is obtained using the resampling approach described above. Guidelines for test selection are shown in Table II.6.2.

Note that if resampling techniques are to be used, it is possible to construct new test statistics to test for a particular type of change – it is not necessary to select test statistics from known tests. Having the flexibility to construct custom test statistics allows great flexibility in what can be tested and for what.

When interpreting test results, it is necessary to remember that no statistical test is perfect, even if all test assumptions are met. Hence, it is recommended to use more than one test. If several tests provide significant results, this provides stronger evidence of change, unless they are very similar, in which case multiple significance is not an additional proof of change.

It is important to examine the test results alongside graphs of the data and with as much historical knowledge about the data as possible. For example, if both step-change and trend results are significant, further information will be needed to determine which of these provides the best description of the change. If historical investigations reveal that a dam was built during the period, and this is consistent with the time series plot, it would be reasonable to conclude that the dam caused a step change.

If test results suggest that there is a significant change in a data series, it is important to try to understand the cause. Although the investigator may be interested in detecting climate change, there may be many other possible explanations, which need to be examined (Kundzewicz and Robson, 2004). It can be helpful to look out for patterns in the results that may indicate further structure, such as regional patterns in trends.

6.2.3 Spatial analysis in hydrology

Hydrological variables may form a spatial-temporal random field, for example, a set of time series of values of a variable for a number of gauges. A spatial field describes discrete observations of a variable in the same time instant in a number of spatial points or remotely sensed data covering the whole area. The spatial aspects of random fields such as rainfall, groundwater level or concentrations of chemicals in groundwater, are important issues in hydrology.

Geostatistics is a set of statistical estimation techniques for quantities varying in space. It lends itself well to applications to random spatial fields, such as precipitation or groundwater quality, thus being applicable to a range of hydrological problems (see Kitanidis, 1992). Geostatistics offers solutions to several practical problems of considerable importance in hydrology. It can be used in interpolation, such as estimating a value for an ungauged location, based on observations from several neighbouring gauges, or plotting a contour map based on scarce information in irregularly spaced locations. It can solve aggregation problems: finding areal estimates based on point observations such as determining areal precipitation from point values. It can aid in monitoring network design, for example, in optimal network extension or, unfortunately more common, optimal network reduction. These applications answer the following question: how to reduce the network while minimizing information loss. By using geostatistics with groundwater flow or transport models, the inverse problem of parameter identification can be solved by determining transmissivity from observed hydraulic head, for example.

A statement of the principal problem of the geostatistical kriging technique can be formulated as looking for a best linear unbiased estimator (BLUE) of a quantity at some unmeasured location x_0 from observations $z(x_1), z(x_2), \ldots, z(x_n)$ in a number of locations $x_1, x_2, \ldots, x_n$:

$$Z(x_0) = \sum_{i=1}^{n} \lambda_i z(x_i) \tag{6.20}$$

where $Z(x_0)$ is the estimator of $z(x_0)$ and λ_i are weights.

Under the so-called intrinsic hypothesis, the estimation variance can be expressed with the help of a mathematical equation containing weights from equation 6.20 and values of a semivariogram. Sets of weights are sought which provide an optimal estimate in the sense that the estimation variance is a minimum. An important advantage of kriging is that it provides, not only the estimated value, but also evaluates the estimation variance. This useful technique originates from mining, where observations are costly, and hence scarce, and optimal

organization of the available knowledge is of primary importance.

Today geostatistics has become an important element of distributed modelling in the geograpical information system environment and an option in interpolation packages.

6.3 MODELLING HYDROLOGICAL SYSTEMS AND PROCESSES

[HOMS J04, K22, K35, K55, L20, L30]

6.3.1 Introduction

The hydrological cycle refers to the circulation of water in the world. It is composed of a number of water fluxes between different water storages. Examples of water fluxes within hydrological processes are liquid or solid precipitation, infiltration, runoff, snowmelt, river flow and evapotranspiration. Examples of the corresponding water storages are atmosphere; land surface such as depressions, ponds, lakes and rivers; vegetation; soil; aquifers and snow cover.

All hydrological processes and hydrological systems have been described by mathematical equations, some of which were derived from rigorous physical laws of conservation of mass and momentum. Others are either of conceptual nature, or a black box type. A comprehensive review of mathematical equations of use in dynamic hydrology can be found in Eagleson (1970). The present section contains a few illustrative examples related to

Table II.6.1. Comparison of parametric and non-parametric tests for change detection, their properties and the assumptions made (after Kundzewicz and Robson, 2004)

Test name	*What it does*	*Properties and assumptions made*
Tests for step change		
Median change-point test/Pettitt's test for change	Test that looks for a change in the median of a series with the exact time of change unknown	Powerful rank-based test, robust to changes in distributional form
Mann-Whitney test/rank-sum test	Test that looks for differences between two independent sample groups, based on the Mann-Kendall test statistic	Rank-based test
Distribution-free CUSUM (maximum cumulative sum) test	Test in which successive observations are compared with the median of the series with the maximum cumulative sum of the signs of the difference from the median as the test statistic	Rank-based test
Kruskal-Wallis test	Tests equality of sub-period means	Rank-based test
Cumulative deviations and other CUSUM tests	Test works on rescaled cumulative sums of the deviations from the mean	Parametric test, assumption of normal distribution
Student's *t*-test	Tests whether two samples have different means – assumes normally distributed data and a known change-point time	Standard parametric test, assumption of normal distribution
Worsley likelihood ratio test	Suitable for use when the change-point time is unknown	Similar to Student's *t*-test, assumption of normal distribution
Tests for trend		
Spearman's rho	Test for correlation between time and the rank series	Rank-based test
Kendall's tau/Mann-Kendall test	Similar to Spearman's rho, but uses a different measure of correlation with no parametric analogue	Rank-based test – extended tests allowing for seasonality exist, for example, Hirsch and Slack (1984) – and autocorrelation
Linear regression	Uses the regression gradient as a test statistic	One of the most common tests for trend, assumption of normal distribution

Note: All the tests make the assumption that the data are identically distributed and independent.

rainfall runoff, flow routing, groundwater, water quality, and snow and ice phenomena.

Hydrological modelling is contributing more and more to integrated models. Beyond simulating hydrological runoff, integrated models include soil erosion, river sediment, ecohydrology, crop yield, and interfaces with other disciplines, such as ecohydrology, climate impact assessment and water management.

6.3.2 Rainfall–runoff relationships

6.3.2.1 General

Rainfall–runoff relationships are used primarily for design, forecasting and evaluation. If streamflow data are unavailable or are too limited for reliable interpretation, rainfall–runoff relationships can be very helpful because of their ability to extract streamflow information from the precipitation records. Because of the relative simplicity and inexpensive nature of the collection of rainfall data, they are generally more abundant than are streamflow data. If a strong relationship can be established between the rainfall and runoff for a catchment of interest, the combination of the rainfall–runoff relationship and the rainfall data may, for example, give more reliable estimates of the frequency of high streamflows than either a regional flood relationship (see Chapter 5) or an extrapolation of meagre streamflow data from the catchment.

In general, rainfall–runoff relationships are developed in two distinct steps: the determination of the volume of runoff that results from a given volume of rainfall during a given time period and the distribution of the volume of runoff in time. The first step is necessary because of the partitioning of rainfall among evapotranspiration, infiltration and runoff (see Volume I, Chapter 4). The second step is required to account for the travel time and the attenuation of the wave of runoff that is generated by the rainfall. Discussion of these two steps constitutes the remainder of this chapter.

6.3.2.2 Runoff volumes

6.3.2.2.1 *Antecedent precipitation index*

The antecedent precipitation index has been developed primarily for river forecasting and is applied over a wide range of drainage areas and conditions. Its derivation for a particular drainage area requires observed rainfall and runoff data over some time interval. It is defined as:

$$I_t = I_o\, k^t + \sum P_i k^{t(i)} \tag{6.21}$$

where I_o is the initial value of the index, k is a recession factor, t is the time interval for the computation, P_i is the number of daily rainfalls that have occurred during the time interval and $t(i)$ is the number of days since each day with precipitation.

It is often convenient to use simplified forms of the antecedent precipitation index. One or more of the variables may have a negligible influence in certain catchments and it is then possible to reduce the number of these variables. However, in all cases the general method is the same.

The effects of vegetative cover, soil type and other important catchment characteristics, as well as the time of year, are reflected in the recession factor. Time of year is expressed as a family of curves representing the seasonal trend of solar energy, vegetative condition and other factors that influence the evaporation and transpiration of moisture in the catchment. The antecedent precipitation index is an expression of the moisture in the catchment and the moisture retention in the soil.

Figure II.6.6 illustrates an example of behaviour of the antecedent precipitation index for a daily

Table II.6.2. Test selection guidelines

Case	*Which test to select*
(a) Data are normally distributed and independent.	This is an unlikely scenario for hydrological data. If applicable, any of the tests listed in Table II.6.1 should be suitable.
(b) Data are non-normal, but are independent and non-seasonal.	Any of the distribution-free tests are suitable. Tests that are based on normality assumption can also be applied by first applying a normal scores or ranks transformation, or by using a relevant test statistic and evaluating significance using resampling techniques.
(c) Data are non-normal, and are not independent or are seasonal.	The data do not meet the assumptions for any of the basic tests given above. It is necessary to extract the test statistic and to evaluate significance levels using block-permutation or block-bootstrap methods.

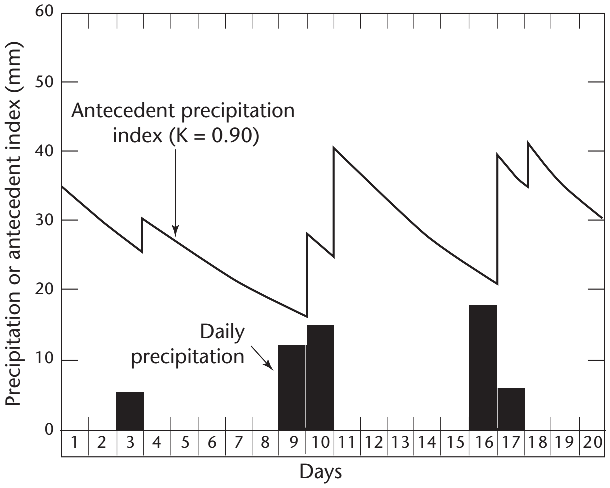


Figure II.6.6. Antecedent precipitation index

recession factor of 0.9. The antecedent precipitation index can be computed from average precipitation for several stations or individually for each station in a drainage area. The latter is often preferable.

Figure II.6.7 illustrates the method of estimating runoff volume from rainfall and the antecedent precipitation index. The dashed lines and arrows demonstrate the use of this diagram. For example, the diagram is entered with a value of 22 mm for the antecedent precipitation index. The long dashes and arrows lead to the month of July and down to storm duration of 24 hours. The example then proceeds to the right to the assumed storm rainfall of 40 mm and up to a runoff of 16-mm average depth over the drainage area.

If the hypothetical storm in the foregoing example had occurred in February, with other conditions being the same, the effect of 22-mm antecedent precipitation would be different. Ordinarily, in February as contrasted with July, the same amount of antecedent precipitation would have left the soil nearly saturated because of dormant vegetation and less evapotranspiration in winter. The short dashes and arrows in Figure II.6.7 show that the runoff from the 40-mm rain in the second example would be 30 mm, hence nearly twice as high as in July.

Frozen ground and accumulations of snow require special consideration in estimating antecedent moisture conditions. With frozen ground, the time-of-year curve that gives the maximum runoff is commonly used. The influence of snow on the ground is properly expressed in terms of the amount and rate of melting, instead of the total accumulation. Snowmelt is discussed in 6.3.5.

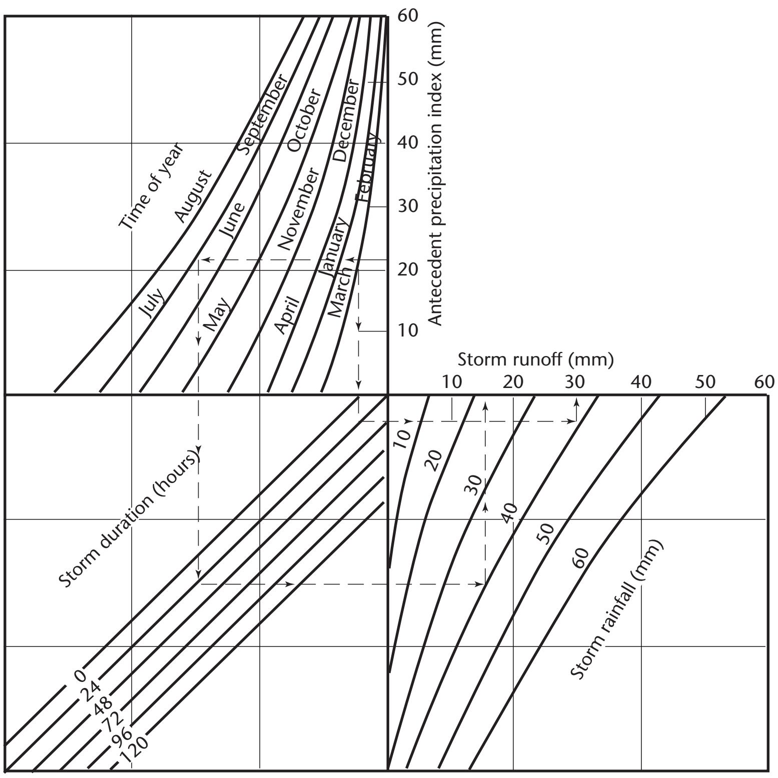


Figure II.6.7. Using the antecedent precipitation index to estimate rainfall runoff

6.3.2.2.2 *Initial base flow as an index to runoff volume*

In humid areas, where streams do not often go dry, base flow in the form of groundwater discharge, at the beginning of a storm is often used as an index of the initial basin conditions. An example of such a relationship is shown in Figure II.6.8. Base-flow discharge reflects conditions throughout the entire area. In some areas, it is found necessary to vary this relationship with season. A common method is to develop one relationship for summer and one for winter, which leads to the inevitable problem of storm events occurring between seasons. The usual solution is to make an estimate of runoff on the basis of each curve and then to interpolate.

The use of initial groundwater discharge as an index to runoff conditions is usually limited to small basins with short times of concentration. In larger areas during a rainy season, one rise of the hydrograph tends to be superimposed on the last, which makes a determination of initial groundwater discharge difficult. The usual approach is to determine initial groundwater discharge for small index basins and to apply them to other nearby areas having similar hydrological characteristics.

6.3.2.2.3 *Moisture-accounting techniques*

Soil moisture deficiency is probably the most important factor involved in the relationship between rainfall and runoff. A practical means of estimating initial soil moisture deficiencies for an area would provide a very useful variable for inclusion in a procedure for correlating storm rainfall to resultant runoff. Instruments for measuring soil moisture for a specific soil profile have become reasonably practical, but the wide variety of soil profiles and moisture conditions that exist in even a small basin makes point measurements of soil

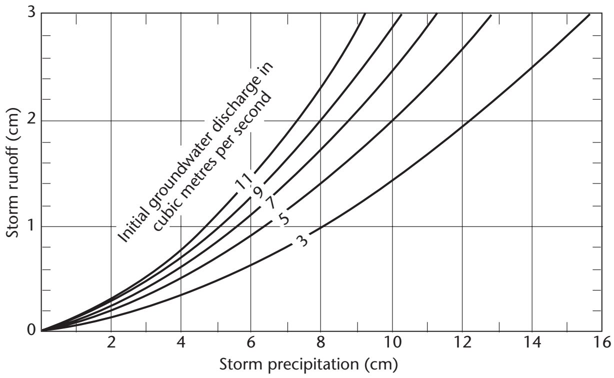


Figure II.6.8. Base flow as an index to rainfall–runoff relationship

moisture of questionable value in a rainfall–runoff relationship.

A more promising approach is the use of an areal accounting technique that results in soil moisture values related to the entire area. In such an approach, precipitation is the inflow and outflow consists of runoff leaving the area by the stream channels plus evapotranspiration into the atmosphere from soil and plant surfaces. The means of estimating the precipitation over the area is the usual problem of deriving spatial averages from point values. Runoff from the area can be determined from streamflow records. The problem becomes one of matching flow to the particular storm that caused it (see 6.3.2). The difference, rainfall minus runoff, is the water that remains in the area and is referred to as recharge, R_c.

The third element, evapotranspiration, is the most difficult to evaluate because its direct measurement is extremely difficult. Most soil moisture accounting techniques are based on the premise that actual evapotranspiration bears a simple relationship to potential evapotranspiration, ET_p, and soil moisture deficiency.

A simple form of soil moisture accounting is one in which the soil profile is considered to have one capacity, S, over the entire area. Soil moisture deficiency, DU_s, is then determined by the following equation:

$$DU_s(t+1) = \begin{cases} 0 & \text{if } DU_s(t) - R_c + ET \leq 0 \\ DU_s(t) - R_c + ET & \text{if } 0 < DU_s(t) - R_c + ET < S \\ S & \text{if } DU_s(t) - R_c + ET \geq S \end{cases} \quad (6.22)$$

where $DU_s(t)$ is the soil moisture deficiency at time t, $DU_s(t+1)$ is the value one time period later, R_c is the recharge resulting from precipitation and/or snowmelt) and ET is the evapotranspiration that occurs between times t and $t+1$. The deficiency varies between the limits of zero and S.

This approach can be made more realistic by multiplying the evapotranspiration by the ratio $(S - DU_s(t))/S$, which acknowledges that actual evapotranspiration decreases along with the supply of available moisture in the soil profile.

Another possible modification would divide the soil profile into layers. In this approach, it is assumed that the upper-layer moisture must first be depleted

before any depletion of the lower layer, and, conversely, recharge to the lower layer is limited to overflow from the upper layer.

The application of soil moisture accounting values in a rainfall–runoff relationship can be made by relating runoff, Q, to discharge computed in the accounting:

$$Q = cQ_U + (1 - c)Q_L \qquad (6.23)$$

where c is a constant, Q_U is the computed runoff from the upper layer, and Q_L is the computed runoff from the lower layer.

6.3.2.2.4 *Temporal distribution of runoff*

To account for the travel time and the attenuation of a volume of water imposed on the catchment by a rainfall event, an accounting through time at the catchment outlet must be performed. This step is usually accomplished by the use of a unit hydrograph, which describes the temporal distribution of runoff leaving the catchment. The unit hydrograph is constrained by the principle of continuity of mass in the following manner:

$$V = \int Q(t)dt \qquad (6.24)$$

where $Q(t)$ is the instantaneous discharge rate, t is time, and V is the runoff volume. The function $Q(t)$ defines a curve whose shape correctly represents the catchment characteristics. To compare hydrographs of different catchments and to assist in the preparation of synthetic hydrographs, deterministic models have been developed that relate the hydrograph characteristics to hydrological and meteorological data. These models are discussed below.

6.3.2.2.5 *Unit hydrograph*

The unit hydrograph for a catchment is defined as the discharge hydrograph resulting from a unit of effective rainfall generated uniformly over the catchment at a uniform rate during a specified period of time. In application, the unit hydrograph is assumed to be time invariant. It is further assumed that events with runoff volumes other than one unit produce hydrographs that are proportional to the unit hydrograph.

6.3.2.2.6 *Derivation from streamflow records*

To determine the volume of runoff from a particular rainstorm, it is necessary to separate the hydrograph into its pertinent components. One component is the direct or storm runoff associated with a particular storm. Another major component is the streamflow persisting from previous contributions to flow. The third major component is the flow from the earlier storms that is delayed by passing through the ground. A portion of that component is known as interflow, that is, water passing through the soil with little delay, and is often included as part of direct runoff. Some of the more recent conceptual models for the continuous simulation of streamflow have provisions for computing each of the above components separately.

This type of analysis does not allow identification of each component by inspecting the observed hydrograph. In less complex methods of analysis in which only two components are recognized, it is possible to separate the observed hydrograph and evaluate the magnitude of the two components. In the following illustration, direct runoff includes both surface runoff and interflow.

One of the simplest of many methods for separating a hydrograph into its major components is illustrated in Figure II.6.9. The trace of base flow is extrapolated (see line segment AB) to the time of peak flow by extending its trend prior to the stream rise. From point B, a straight line is drawn to intersect the hydrograph at point C a fixed time later. The time in days from B to C is determined largely by the size of the drainage area. It is generally about $(A/2)^{0.2}$, where A is drainage area in square kilometres.

Several methods of hydrograph separation are commonly used. The same technique should be used in both application and development, a requirement that is more important than the method, however.

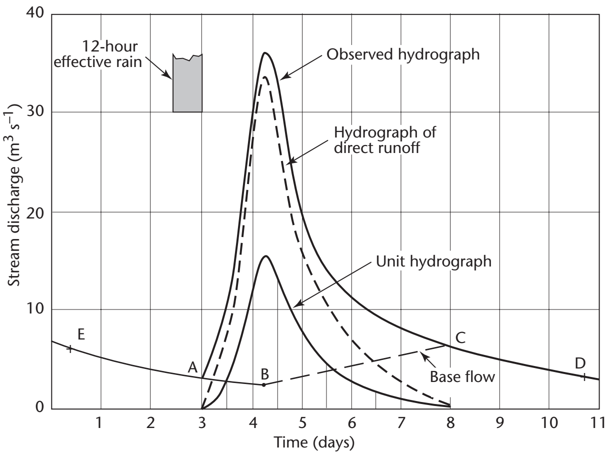


Figure II.6.9. Hydrograph analysis

The total runoff associated with a particular storm or stormy period can be determined by the following method. In Figure II.6.9, the area under the hydrograph between times A and D is the storm runoff because the beginning and ending points represent the same groundwater recession conditions and, therefore, the same storage.

Figure II.6.9 illustrates the essential steps for deriving a direct runoff unit hydrograph from observed data. These steps may be performed either graphically or numerically. The hydrograph of direct runoff is the flow in excess of the trace ABC. The volume of direct runoff is obtained by integrating the area under the hydrograph. If a planimeter is not available, a convenient method is the counting of squares. In this hypothetical example, the volume of direct runoff is found to be 4 320 000 m^3. Over an assumed drainage area of 200 km^2, this volume represents an average depth of 2.16 cm. To obtain the unit hydrograph, it is necessary to divide each ordinate of the direct runoff hydrograph by 2.16. The hydrograph thus determined shows the shape of the hydrograph that would result from one centimetre average depth of direct runoff over the drainage area, that is, the unit hydrograph.

In the records of some catchments, it is difficult to find unit or single storms that produce stream rises uncomplicated by other events. In such cases, the derivation of a unit hydrograph becomes more complex. One method of deriving a unit hydrograph under these circumstances is to assume an initial unit hydrograph, and to reconstruct the hydrographs of direct runoff for several storms using estimated runoff increments and to refine the unit hydrograph by successive approximations as indicated by the results. This reconstruction method is shown in Figure II.6.10 and by:

$$q_n + Q_nU_1 + Q_{n-1}U_2 + Q_{n-2}U_3 + \ldots + Q_{n-i+1}U_i + \ldots + Q_1U_n \qquad (6.25)$$

where q_n is the rate of discharge from direct runoff at time n, U_i is the i-unit hydrograph ordinate and Q_{n-i+1} is the direct runoff for the i^{th} interval. This equation can also be used as the regression model for unit hydrograph derivation by least squares.

For drainage areas of 200 to 2 000 km^2, time increments of six hours are commonly used for unit hydrograph development, but for higher accuracy, shorter time intervals may be employed. Smaller drainage areas may also require shorter time increments. The time increments should be small enough to give good definition of the hydrograph shape and allow a forecast to be made before too large a time increment has elapsed. For drainage areas larger than about 2 000 km^2, unit hydrographs of larger time increments may be used, but as a rule, unit hydrographs should be applied to tributary areas and may be combined by routing.

As might be expected from considerations of channel hydraulics, there is a tendency for the peakedness of unit hydrographs to increase with the magnitude of runoff. Accordingly, in practical applications, a family of unit hydrographs may be used for a particular catchment area, with higher peaked unit hydrographs for the cases with large amounts of runoff and flatter peaks for the lesser amounts of runoff. Often only two categories comprise the family.

Skill in the use of unit hydrograph is acquired from study and practice. For other methods than those described in this section, and for refinements, reference may be made to textbooks and handbooks of agencies that routinely use unit hydrographs in their regular operations.

6.3.2.2.7 *Derivation by synthetic methods*

It is often necessary to plan constructions or operations for ungauged streams. In such cases, it is helpful to develop synthetic unit hydrographs (Dooge, 1973). A commonly used derivation of a unit hydrograph is the procedure derived by Snyder in which a large number of basins and unit hydrographs were analysed to derive relationships between the shape of the unit hydrograph and the

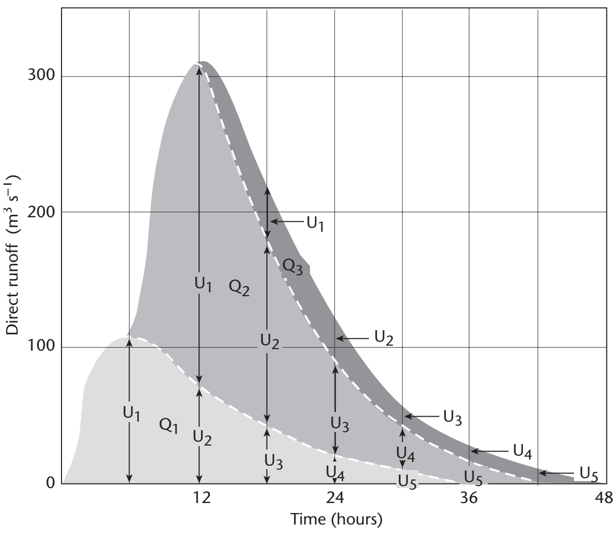


Figure II.6.10. Reconstruction of direct runoff hydrograph

objective physical characteristics of the drainage basin.

The important parameters in the shape of a unit hydrograph are its peakedness, the length of its base and the basin lag, which may be defined in various ways; here, however, it is the time from the centroid of rainfall to the peak of the hydrograph. In Snyder's method, the basin lag, t_p, is given in hours as:

$$t_p = C_1(ll_c)^n \tag{6.26}$$

where C_1 converts units and is an empirical coefficient, l is the length of the main stream in kilometres, l_c is the distance in kilometres from the centroid of the drainage area to the outlet and n is an empirical exponent.

For peakedness of the unit hydrograph, this method uses a standard duration of rain, t_p/C_2, with C_2 being derived empirically. For rains of this duration:

$$Q_p = C_3A/t_p \tag{6.27}$$

where Q_p is peak rate of runoff in m^3 s^{-1}, C_3 is an empirical constant, A is drainage area in km^2 and lag t_p is in hours. The time base in days T_b is as follows:

$$T_b = d + C_4t_p \tag{6.28}$$

The constants d and C_4 are fixed by the procedure used to separate base flow from direct runoff.

For durations T_R other than the standard duration of rain, the corresponding lag, t_c, is the following:

$$t_c = t_p + f\,(T_R) \tag{6.29}$$

here $f(T_R)$ is a function of duration.

Snyder's coefficients were derived for streams in the Appalachian Mountains of the United States. The general method has been found applicable in other regions, but different coefficients are to be expected for different types of topography, geology and climate.

Rodriguez-Iturbe and Valdes (1979) developed a physically based methodology for synthesizing an instantaneous unit hydrograph with the help of empirical laws of geomorphology and climatic characteristics. They proposed the geomorphologic instantaneous unit hydrograph, later known as the geomorphoclimatic instantaneous unit hydrograph. They also developed equations for the value of peak and time to peak of the geomorphologic instantaneous unit hydrograph as functions of the bifurcation ratio, length ratio, area ratio, length of highest order stream and flow velocity.

6.3.2.2.8 *Conversion of unit hydrograph durations*

A suitable rainfall of unit duration is rarely observed. Variations of rainfall in time and space produce different hydrographs, though the total amount and duration of the rain may be exactly the same. Thus, the derivation of a general unit hydrograph requires an averaging of several unit hydrographs.

One technique for generalizing unit hydrographs is by comparison of unit hydrographs of different durations. If a unit hydrograph of duration t hours is added to itself, lagged t hours, and the ordinates divided by two, the result is a unit hydrograph for $2t$ hours. Similar conversions are evident.

A broader application of this basic idea for manipulating unit hydrographs is known as the summation or S-curve method. The S-curve is the hydrograph that would result from an infinite series of runoff increments of one centimetre in t hours. It is constructed by adding a series of unit hydrographs, each lagged T hours with respect to the preceding one. With a time base of T hours for the unit hydrograph, a continuous rain producing one centimetre of direct runoff per t hours would develop a constant outflow at the end of T hours. Thus, T/t hours would be required to produce an S-curve of equilibrium flow.

Construction of an S-curve can be accomplished by a numerical, rather than a graphical, procedure. A unit hydrograph for any duration t can be obtained by lagging the S-curve t hours and obtaining ordinates of lagged and unlagged S-curves. To obtain unit volume, these ordinates must be multiplied by the ratio of the duration of the original unit hydrograph to t hours.

The instantaneous unit hydrograph is the unit hydrograph whose time unit, t, is infinitely small. Construction of a t-hour unit hydrograph from an instantaneous one is performed by means of an S-curve.

6.3.2.2.9 *Isochrone method*

The isochrone method is an expression of one of the first concepts of runoff from a basin. The runoff

from different portions of a drainage basin arrives at a point in the stream at different times. The first water to leave the basin during a stream rise generally comes from the area nearest the catchment outlet. Later, water comes from larger areas in the central portion of the basin, and finally, water comes from remote portions of the drainage area. Thus, the drainage basin may be divided into zones from which the water arrives sequentially at the measurement point. The lines dividing these zones in Figure II.6.11(a) are called isochrones. The distribution of the isochronal areas, the time-area distribution, is considered to be constant for a given basin for all flood hydrographs.

To compute this distribution, it is necessary first to compute or assume an average travel time or average velocity of streamflow. The isochrones are drawn on a map of the basin according to the average velocity of flow in the channel or average travel time. The area of each zone is then determined by using a planimeter and the values are plotted against the corresponding time lag (Figure II.6.11(b)).

The time-area distribution is indicative of the hydrograph for uniform rainfall of unit duration, Δt, the time difference between isochrones. If there are several periods of rainfall, each resulting in varying quantities of runoff over the different zones:

$$Q_t \Delta t = A_1 V_t + A_2 V_{t-1} + A_3 V_{t-2} + \ldots + A_c V_{t-c+1} \qquad (6.30)$$

where Q_t is the average discharge during the period, Δt, ending at time t, A is the time-area histogram ordinate at that period and V_t is the zonal runoff during the same period. Care must be taken to ensure consistent units. Figure II.6.11(c) illustrates the computation of the resultant hydrograph with three periods of uniform runoff from the catchment.

The resultant hydrograph reflects the lag characteristics of the catchment. Since the actual hydrograph would be affected by channel storage, the hydrograph computed from equation 6.30 should be routed through storage. Any of the several routing techniques described in the literature can be used. Two such techniques are described in 6.3.5. It is usually found to be advantageous to adjust the isochrones and routing parameters by trial and error to obtain the best combination for simulation of observed hydrographs.

The isochrone method allows non-uniform distributions of rainfall to be taken into account when there are enough raingauges in the basin to delineate the rainfall pattern reliably. This is an advantage over the unit hydrograph described previously.

6.3.3 Groundwater modelling

6.3.3.1 General modelling considerations

Groundwater is increasingly becoming an important source of water for mankind as surface water resources become more depleted through the increased effects of abstraction and pollution.

Because groundwater is a concealed and obscure asset, its conservation and management is expensive and scientifically challenging owing to the lack of evidence, knowledge and understanding of its location, quantity and character. Consequently, in order to assess its extent, volume and quality and in turn develop, manage and protect it, it is necessary to construct representative modelled scenarios to examine its potential and reliable volume and quality. This section provides a summary of the science and methods employed in hydrogeological modelling within a practical context of which the principal

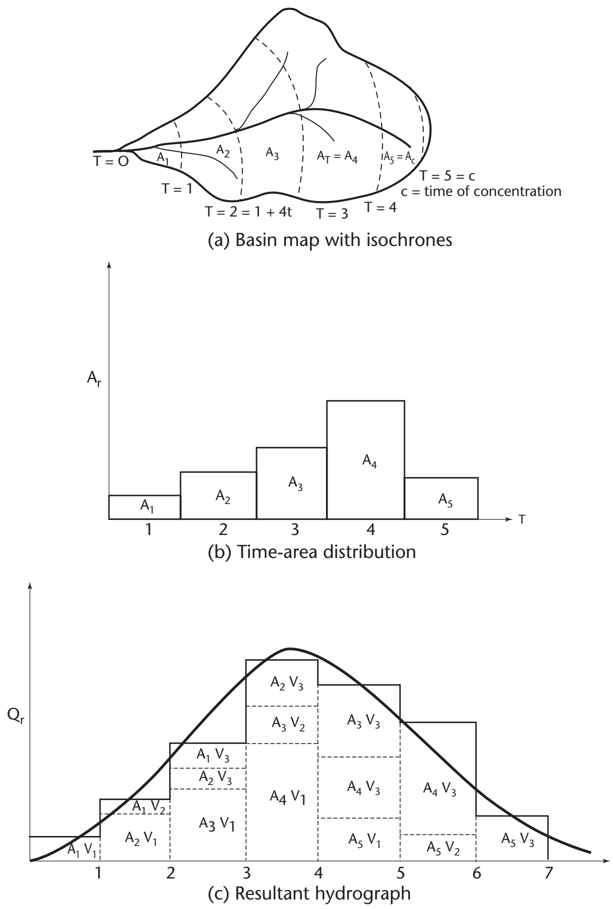


Figure II.6.11. Isochrone method

elements are process, development, control and conservation.

6.3.3.2 Development of a conceptual model

To adequately represent a hydrogeological regime, there are a significant number of characteristics that must be replicated by using a model. These model elements comprise several representations for consideration (Bear, 1980, 1988).

The type and detail of the conceptual model will depend on the scale, amount of time and resources – availability of data, technical expertise, staff resources, computing facilities – assigned to the task, as well as the quality of the decision-making process, professional risks and legal and statutory framework.

Conceptual modelling is continuous and cyclical; therefore, a tiered approach from basic to intermediate to detailed is appropriate. The assumptions included in the conceptual model should relate to the issues described below.

6.3.3.3 Development of a mathematical model

The principal elements of the model include the following components:
(a) A definition of the geometry of the surfaces that bound the domain;
(b) Equations that express the balances of the components, for example, mass of fluids, mass of chemical species and energy;
(c) Flux equations that relate the fluxes of the components to the relevant variables of the problem;
(d) Constitutive equations, which define the behaviour of the particular phases and chemical species involved, for example, dependence of density and viscosity on pressure, temperature and solute concentration;
(e) Sources and sinks, often referred to as forcing functions, of the component quantities.

In terms of the modelling runs, the settings comprise the following states:
(a) Initial conditions that describe the known state of the system at some initial time;
(b) Boundary conditions that describe the interaction of the considered domain with its environment, that is, outside the delineated domain, across their common boundaries.

If a new numerical model and its associated code must be used to solve the mathematical model that is being employed, a strict verification procedure should be undertaken to check that it is fit for purpose through previous proven applications. If practical, comparative scenarios should be run using different codes.

The groundwater regime is controlled by geological and climatic conditions and is exploited by man to meet the needs of water development while the environmental requirements are met through the residual balance. To assess the presence, extent and variability of available groundwater resources, a range of investigation and testing mechanisms have to be undertaken. These draw on a wide skills base encompassing a range of Earth sciences, including hydrometeorology, hydrology, pedology, geomorphology, petrology, geology and water chemistry.

Groundwater constitutes a portion of the Earth's water circulatory system known as the hydrological cycle with water-bearing formations of the Earth's crust acting as conduits for the transmission and as reservoirs for the storage of water. Water enters these formations from the ground surface or from bodies of surface water, after which it travels slowly for varying distances until it returns to the surface by the action of natural flow, plants or man. The storage capacity of groundwater reservoirs combined with slow flow rates can provide large and extensively distributed sources. Groundwater emerging into surface water stream channels aids in sustaining streamflows when surface runoff is low or non-existent. Similarly, water pumped from wells in many regions represents the sole water source in many arid areas during much of the year.

Water within the ground moves downwards through the unsaturated zone under the action of gravity, whereas in the saturated zone it moves in a direction determined by the hydraulic situation. The principal sources of natural recharge include precipitation, streamflows, lakes and reservoirs. Discharge of groundwater occurs when water emerges from underground. Most natural discharge occurs as flow into surface water bodies, such as streams, lakes and oceans, and flow to the surface appears as a spring. Groundwater near the ground surface may return directly to the atmosphere by evaporation from within the soil and by transpiration from vegetation. Pumpage from wells constitutes the major artificial discharge of groundwater.

Groundwater occurs in permeable geological formations known as aquifers that have a structure facilitating the flow of water to take place under natural conditions with aquicludes being impermeable formations that preclude the

transmission of water movement. The portion of a rock or soil that is not occupied by solid mineral matter may be occupied by groundwater (Todd, 2005). These spaces are known as voids, interstices, pores or pore spaces and are characterized by their size, shape, irregularity and distribution. Original interstices were created by geologic processes governing the origin of the geologic formation and are found in sedimentary and igneous rocks. Secondary interstices developed after the rock was formed and include joints, fractures and solution openings. The porosity of a rock or soil is a measure of the contained interstices and is expressed as the percentage of the void space to the total volume of the mass. If a is the porosity, then:

$$a = 100w/V \tag{6.31}$$

where w is the volume of the water to fill, or saturate all of the pore space, and V is the total volume of the rock or soil.

Porosities can range from zero to 50 per cent, depending on the shape and arrangement of the individual particles, size distribution and degree of compaction and cementation.

There are a number of models that are used to represent groundwater movement and transport phenomena. They include the following:

(a) A physical representation using a scaled model comprising a medium through which a fluid is introduced and monitored by pressure and head instrumentation;
(b) An electrical representation in which head, flow and conductivity are represented by voltage, current and resistance;
(c) A mathematical representation using a set of algorithms to represent the principal processes;
(d) A stochastic analysis to characterize subsurface flow and transport modelling.

In practice, most hydrogeological modelling currently used falls under (c) and (d) above. With regard to transport phenomena relating to groundwater contamination in which two- and three-phase flow conditions occur, the use of mathematical models is regarded as essential because of the complex scenarios that have to represented and analysed.

The movement of groundwater in its natural state is governed by established hydraulic principles. The flow of water through aquifers can be expressed by a law derived by Darcy in 1856 that states that the flow rate through a porous media is proportional to the head loss and inversely proportional to the length of the flow path. Darcy's law may expressed in general terms as:

$$Q = KA\ dh/dL \tag{6.32}$$

where Q is the flow rate, K is the coefficient of permeability (sometimes referred to as hydraulic conductivity) and dh/dL is the hydraulic gradient. This relationship is shown in Figure II.6.12.

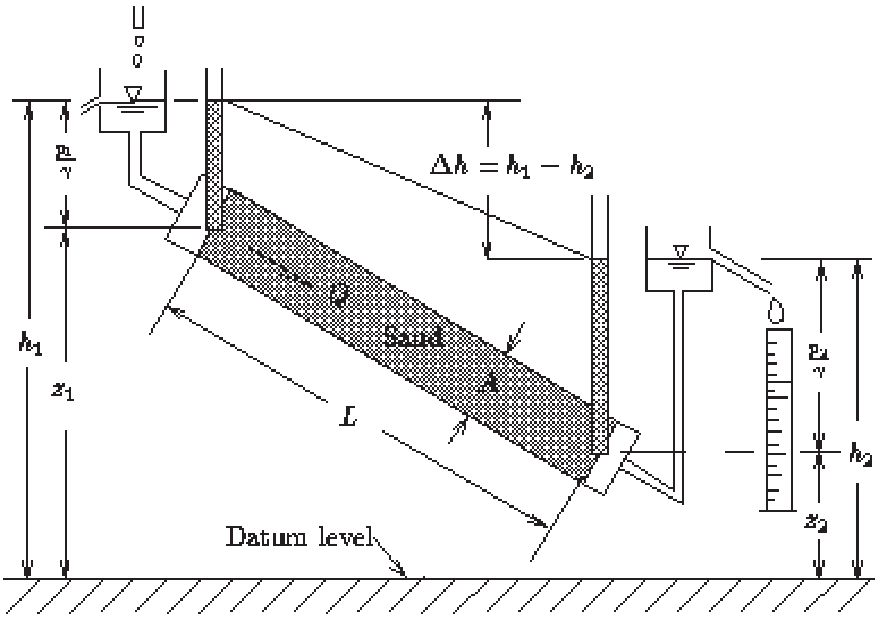


Figure II.6.12. A representation of Darcy's experiment

With the nomenclature of this figure, Darcy's law takes the following form:

$$Q = K\,A\,\frac{h_1 - h_2}{L} \qquad (6.33)$$

where, h (dimension: $[L]$) is the piezometric head:

$$h = z + \frac{p}{\rho g} \qquad (6.34)$$

where z is the elevation of the point at which the piezometric head is being considered above some datum level, p and ρ are the fluid's pressure and mass density, respectively, and g is the gravity acceleration.

The hydraulic conductivity, K, can then be expressed as follows:

$$K = k\,\frac{\rho g}{\mu} = \frac{kg}{v} \qquad (6.35)$$

where g is the gravity acceleration, and k (dimension: $[L^2]$) is the permeability or intrinsic permeability of the porous medium. It is a coefficient that depends solely on the properties of the configuration of the void space.

Groundwater flow is an important aspect of hydrogeology and is based on the principles that govern the flow of fluids through porous media. It requires a broad knowledge of fluid mechanics but cannot be adequately described within this brief outline of hydrogeology. However, the principal subdivisions of groundwater flow can be summarized according to the dimensional character of the flow, the time dependency of the flow, the boundaries of the flow region or domain and the properties of the medium and the fluid.

6.3.3.4 Model operational options

Different options can be selected when designing a model:

(a) The dimensionality of the model (one, two, or three dimensions);
(b) Steady state or time-dependent behaviour;
(c) The number and kinds of fluid phases and the relevant chemical species involved;
(d) The possibility of phase change and exchange of chemical species between adjacent phases;
(e) The flow regimes of the fluids involved, for example, laminar or non-laminar;
(f) The existence of non-isothermal conditions and their influence on fluid and solid properties and on chemical-biological processes;
(g) The relevant state variables and the areas or volumes over which averages of such variables should be taken.

All groundwater flow in nature is to a certain extent three dimensional but the difficulty in solving groundwater flow problems depends on the degree to which the flow is three dimensional. It is practically impossible, however, to analyse a natural three-dimensional flow problem unless it can be expressed in terms of a two-dimensional problem that assumes that a degree of symmetry exists. Consequently, most solutions are based on assuming that the problems being analysed are two dimensional or have special symmetry features.

In general, groundwater flow is evaluated quantitatively based on a knowledge of the velocity, pressure, density, temperature and velocity of water percolating through a geological formation. These water characteristics are often unknown variables and may vary in space and time. If the unknown or dependent variables are functions of only the space variables, the flow is assumed to be steady; if the unknowns are also functions of time, the flow is considered to be unsteady or time dependent.

The flow of groundwater in the space made up by the water-filled pores – the aquifer – is dependent on the bounding surfaces of the medium, boundaries. If these boundaries are fixed in time and space for different states of flow, the aquifer is confined. However, if it possesses a free surface that varies with the state of the flow, it is unconfined.

Groundwater flow in the aquifer is controlled by the nature, properties and isotropy of the medium. If the medium's properties at any one point are the same in all directions from that point, it is considered to be isotropic; and if not, it is considered to be anisotropic. The medium is considered to be of heterogeneous composition if its nature, properties or conditions of isotropy or anisotropy vary from point to point in the medium, and homogeneous if its nature, properties and isotropic or anisotropic conditions are constant over the medium.

Another subdivision is that of saturated and unsaturated flow. Flow is saturated if the voids of the medium are completely filled with fluid in the phase of the main flow. The flow is unsaturated if this is not the case. Deep percolating groundwater flow is always saturated, whereas above the saturated medium in the absence of overlying impermeable strata there is a zone of unsaturation. The boundary between these two zones is called the water table or the phreatic surface. The latter zone is occupied partially by air and partially by water and is referred to as the unsaturated zone or the

zone of aeration. It is comprised of an upper zone, which is referred to as the soil water zone, an intermediate zone and a lower zone, which is known as the capillary zone.

Water in the soil water zone exists at less than saturation with a soil moisture deficit occurring, except when excessive water enters it as a result of prolonged rainfall. The zone extends from the ground surface through the major root zone; its thickness depends on soil type and vegetation.

The three principal hydraulic properties of an aquifer are porosity, which determines the volume stored, specific yield that controls the volume that it yields either through natural drainage or when it is pumped, and permeability, which governs the rate that water flows through it.

A range of approaches to groundwater modeling are contained in the literature. The most common are physically based methods originating from rigorous physics of flow in porous media. However, conceptual, and even black box methods are also used.

The mathematics of flow and transport in unsaturated and saturated porous media are relatively complex. There are a variety of physically based equations, developed for different assumptions, and simplifications for a variety of configurations, including confined, leaky, and unconfined aquifers. A classical review of physically based approaches can be found in Eagleson (1970). See also Maidment (1992).

The process of groundwater flow is governed by non-linear partial differential equations in three dimensions, expressing conservation of mass, or continuity, conservation of momentum and a state equation. Under the assumption of negligible compressibility of water and the porous medium, the isothermal unsteady laminar flow, with no sources or sinks of water, can be described by the following partial differential equation (see Eagleson, 1970):

$$W + S_s \frac{\partial h}{\partial t} = \frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right) \tag{6.36}$$

where h is the piezometric head, K_x, K_y, and K_z are the hydraulic conductivities (for isotropic mediums, $K_x = K_y = K_z = K$), W is a general term for sources and sinks of water, and S_s is the specific storage. In the variably saturated flow formulation, the unsaturated hydraulic conductivity and specific moisture capacity are functions of moisture content.

An equation describing transport processes in groundwater can be developed by using equation 6.51. The advection-dispersion equation of transport is used to simulate quality aspects of groundwater flow, such as transport of solutes, in conservative and reactive cases.

Physically sound partial differential equations of groundwater flow and transport form a core of distributed groundwater models in which the solution is commonly achieved by finite-difference or finite-element techniques. A review of computer software for solving subsurface water problems has been carried out by Anderson and others (1992). Among the codes included in HOMS component L20.2.04 is the modular finite-difference groundwater flow, or MODFLOW, model, a versatile software package developed by the United States Geological Survey, which has been commonly used worldwide in numerous applications.

The MODFLOW model (McDonald and Harbaugh, 1988) simulates groundwater flow in a porous medium in three dimensions, as well as modelling flow in two dimensions. A modular structure was used for the program and documentation in order to make the model easier to understand and modify when necessary. The code consists of a series of packages or modules that can be selected for a problem at hand. Modules include those for equations solvers, stream, recharge, pumping and evapotranspiration. The application area of MODFLOW includes steady-state and transient groundwater flow, groundwater flow in confined, leaky and unconfined aquifers and a number of special flow problems such as spring flow and flow to a well. Wells, rivers, drains, evapotranspiration and recharge can be simulated and are represented as head-dependent sources or sink terms in which the head outside the model is user specified. MODFLOW can be used in studies of interactions between groundwater and surface water interactions, such as flow to partially penetrating rivers and lakes. Aquifer hydraulic parameters, boundary conditions, initial conditions and stresses are required model input. The input is from text files with the data laid out in a prescribed order and format. The input data must correspond to the specified grid structure. The primary model output is the head at each model node. In addition, a water budget is calculated, and the flow through each model cell can be stored in a disk file. MODFLOW is probably the most widely used groundwater model in the world.

The MODFLOW package is designed for use by experienced groundwater hydrologists. Useful pre-processors and post-processors are available that reduce the user's efforts.

6.3.3.5 Planning

To undertake a groundwater modelling project, the first stage – and a very essential one – is to define the purpose of the work. For larger projects, this may require an initial scoping study to define the requirements and carry out an analysis that will identify the aims of the project, and review previous studies and available data. As well as defining the objectives of the project and the principal tasks to be undertaken, scoping should also define the main outputs anticipated for the work.

Planning also involves the identification of the type of information the model is expected to provide to make management decisions and of the data that is available or will have to be derived by establishing a monitoring programme. Additionally, it is essential to determine the available resources, including expertise, skilled personnel, monitoring equipment, field data and computers, that are required to construct and utilize the model within the budget constraints that can be identified. This includes the ability to understand and describe processes that take place and the data required for validating the model and determining the numerical values of its coefficients. Consideration should also be given to the local legal and regulatory framework which pertains to the case under consideration to ensure that the model results will be sufficiently robust, extensive and detailed to satisfy future scrutiny.

It is good practice to set up a management team, which should include relevant stakeholders to guide the project, review interim outputs, resolve differences of opinion and reach an agreement on the acceptability of each stage of the model development process.

Having determined the objectives of the modelling project, a phased approach is required because of the range of uncertainties and the relatively high costs and long work programme that is generally associated with groundwater modelling. Recognizing these issues, the Environment Agency of England and Wales produced a guideline (Environment Agency, 2002) setting out the sequential phases that should be considered in the groundwater modelling process, shown in Figure II.6.13.

The approach illustrated above comprises a decision support system that progresses from a scoping study, to a conceptual model and a historical model, ending with a predictive model that can subsequently be refined with operational data. These means are designed to meet the objectives, assess the options, derive a response to options, evaluate the results, select a preferred solution and set up a monitoring system to assess the outcomes.

6.3.4 Snowmelt models

Snowmelt is analogous to rainfall with respect to the supply of water for infiltration and runoff, except for the lag of the melted snow in the snow cover. In some areas, snowmelt water is the principal contributor to reservoirs, rivers, lakes and aquifers. In mountainous snow regions, snowmelt becomes an important component of runoff, usually making up more than 50 per cent of the total streamflow. In some mountain basins, snowmelt makes up 95 per cent of the runoff.

Ordinary measurements of incremental changes in water equivalent of the snow cover are not satisfactory measurements of snowmelt, largely because of the inherent observational and sampling errors. Two additional and compelling reasons exist for estimating, rather than observing, snowmelt. One is in forecasting streamflow, where it is advantageous to forecast the causes of melt instead of merely waiting for the resulting melt. The other reason, particularly for design and planning, is the need to extrapolate extreme melting rates on the basis of physical processes. Snowmelt has been incorporated in a number of hydrological models as indicated in the short review of HOMS components in 6.1.6.

In principle, a conceptual snowmelt runoff model is the coupling of a routine for snow accumulation and ablation with a rainfall-runoff model. The joint model can be used in all climatic conditions for year-round forecasting. Snowmelt runoff models have also been developed specifically for use for the spring snowmelt period. In all cases, melting of a snowpack is driven by the energy balance. Conservation of energy dictates that the change in snow temperature is balanced with the energy fluxes entering or leaving the pack. The conservation of mass within a snowpack can be described by the following simple continuity equation:

$$I - O = \frac{dS}{dt} \tag{6.37}$$

The inputs are precipitation, condensation and freezing surface water, while the outputs are

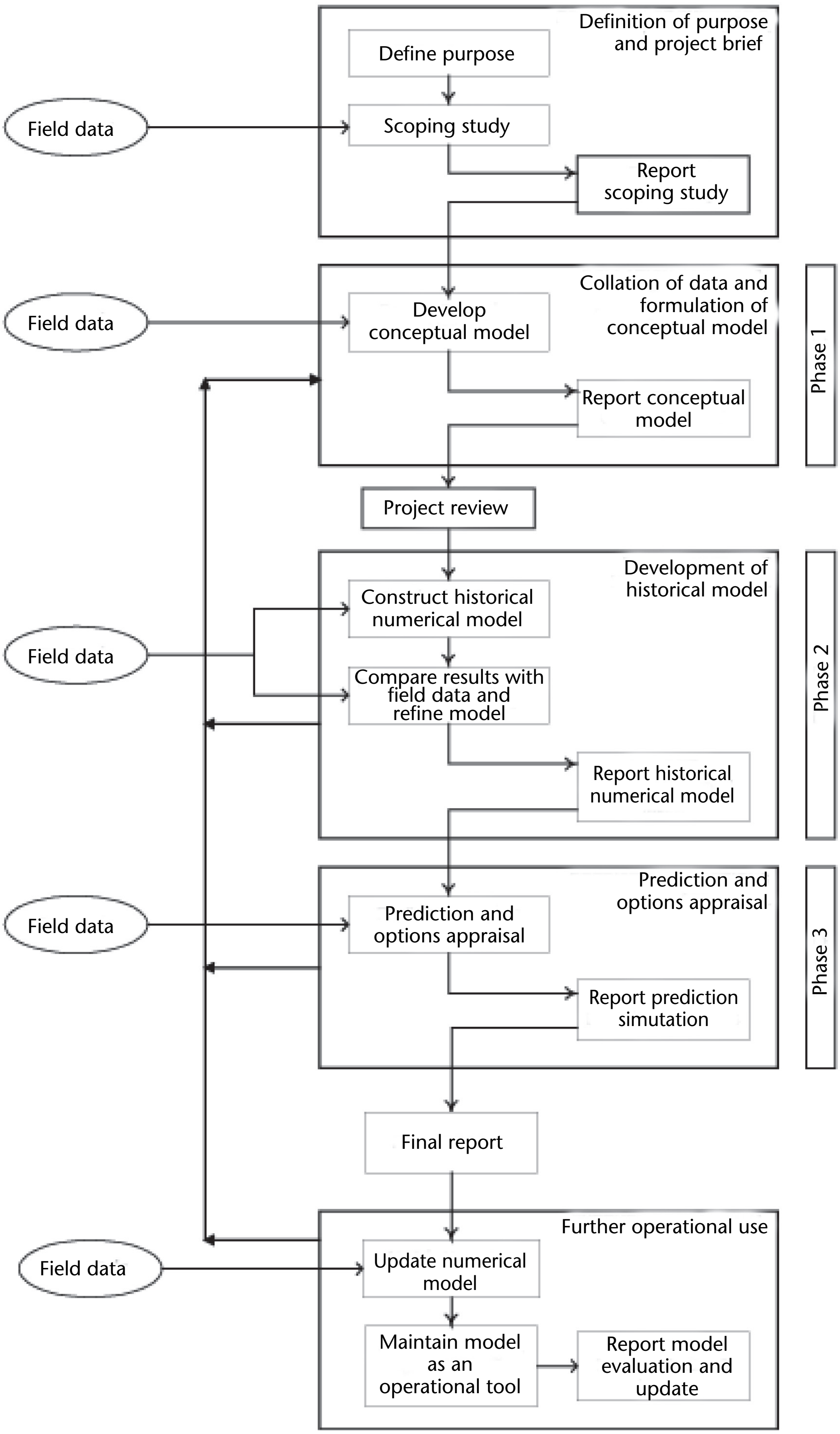


Figure II.6.13. Groundwater modelling process (Environment Agency, 2002)

sublimation and runoff, with all units expressed typically in millimetres of water. Mass changes to the snowpack can also occur through blowing snow, sublimation and accumulation (Pomeroy and Brun, 2001) and snowfall on the ground can be influenced by canopy interception, which can also represent a significant loss of snow mass. Some of the intercepted snow may also blow off the trees and ultimately back to the ground, but much of it may sublimate and be lost to the snowpack (for further details on snow canopy interception, please refer to Hedstrom and Pomeroy, 1998).

Because of these complexities, conceptual methods for describing physical properties of snow and subsequent melt at the catchment scale have been developed. These include snowcover depletion curves and temperature index melt models, both of which are used in many operational hydrological modelling systems to describe and predict the hydrological response of snow-covered catchments.

6.3.4.1 Index methods for estimating basin runoff

Many medium- and long-term snowmelt streamflow forecast models are based on statistical index methods. Available data on precipitation and snowcover in the mountains do not, as a rule, make it possible to determine the amount of snowpack on the ground and may serve only as an index of this value. For this reason, relationships between seasonal flow and a snow-accumulation index are of a statistical nature. Although suitable for forecasting purposes, they cannot be used for water-balance analyses in most cases.

The success of a long-term forecast depends very much on how well the snow-accumulation index represents the actual conditions. There are at least five additional factors that may have some influence on runoff, and consequently on the correlation between the runoff and the snow-accumulation index:
(a) Antecedent groundwater storage;
(b) Amount of precipitation occurring between the last snow survey and the issuing date of the forecast;
(c) Amount of precipitation during the snowmelt period or the period for which the forecast is issued;
(d) Amount of sublimation of the snowpack between the last survey and the issuing date of the forecast;
(e) Amount of sublimation of the snowpack during the snowmelt period or the period for which the forecast is issued.

In those river basins where base flow from aquifers represents a substantial proportion of the total runoff and varies considerably from year to year, the accuracy of the correlation can be increased by taking antecedent groundwater conditions into account.

Precipitation can be taken into account in two ways:
(a) By combining a precipitation index with the snow-accumulation index or using the sum of these indices as a single variable;
(b) By using a precipitation index as a supplementary variable.

Subsequent precipitation should be included in the runoff relationship during procedure development. This ensures that the precipitation effects are included in deriving the statistical snowmelt forecasting relationships.

If financial budgets allow, snow surveys should be conducted in the mountains several times during the winter so that snow-accumulation trends can be derived. The final snow survey is generally carried out at the end of the snow-accumulation period just before the beginning of the spring snowmelt. Snow-survey data at the end of the snow-accumulation period are used for calculating the snow-accumulation index.

Snow courses located at various altitudes are used to obtain data to establish a relationship between the snow water equivalent and the altitude, $w = f(z)$. A different relationship is obtained for each year. When the observation data are insufficient for plotting graphs of $w = f(z)$, the multiple correlation between runoff and the snow water equivalent at each point of observation can be used. Snow course data may still be used as input to statistical models to forecast runoff.

In most cases, the best index of the water available for runoff from mountainous areas can be developed from a combination of precipitation and snow-survey data. This can be accomplished by statistical approaches.

6.3.4.2 Conceptual snowmelt runoff models

Catchment runoff can be estimated using a number of possible algorithms that represent the physics of a melting snowpack. In many ways, melted snow is treated the same as rainfall and infiltrated into the soil matrix using a number of possible infiltration algorithms. Snowmelt runoff simulation models generally consist of a snowmelt model and a

transformation function. The snowmelt model generates liquid water from the snowpack that is available for runoff and the transformation model is an algorithm that converts the liquid output at the ground surface to runoff at the basin outlet (Donald and others, 1995). The snowmelt and transformation models can be lumped or distributed in nature. Lumped models use one set of parameter values to define the physical and hydrological characteristics of a watershed. Distributed models attempt to account for spatial variability by dividing the basin into sub-areas and computing snowmelt runoff for each sub-area independently using a set of parameters corresponding to each of the sub-areas. Snowmelt models generally include a snowcover representation which can range from a simple single-layered snowpack (see, for example, Anderson, 1973) to a multi-layer conceptual snowpack, as illustrated by Brun and others (1992). Snowpack representation has implications for the timing of the snowmelt runoff because of its ability to store water.

Many operational snowmelt runoff models use some form of a temperature index or a degree-day method to determine when snowmelt occurs and how much snowmelt may occur in a specific period of time. Snow-accumulation and ablation models use temperature and precipitation to accumulate the snowcover and air temperature as the sole index to the energy exchange across the snow-air interface. The latter aspect is usually modelled using the degree-day method, which uses air temperature as the index of snowcover outflow. The degree-day method does not explicitly account for those processes that cause snowcover outflow to differ from snowmelt, that is, refreezing snowmelt caused by a heat deficit and retention and transmission of liquid water. A diagram of the model developed by Anderson (Anderson, 1973) is shown in Figure II.6.14. Actual measurements of snowcover from snow surveys or point measurements may be used as an additional source of information to improve the seasonal volume forecasts from conceptual models that use only temperature and precipitation as input (Todini and others, 1978).

6.3.4.3 Extended streamflow modelling

Conceptual models can only simulate snowmelt runoff for the period for which input data are available. Forecasts for the future can be made by using forecast values of precipitation and temperature derived from statistical or stochastic analysis or from extended predictions using numerical weather models. The pattern of the seasonal runoff cannot be forecast satisfactorily unless the effects of future weather conditions are taken into account.

For index and statistical forecast procedures, this can be accomplished by using indices for the rest of the season based on past records of precipitation and temperature. For conceptual models, climatological data for many years, generally 20 or more, should be used to develop hypothetical runoff sequences for each year's conditions. Probability distributions may be developed from these simulations for any specific period of time in the future and for a specific hydrological characteristic, such as peak flow, volume or discharge per unit area (Twedt and others, 1977). This pre-supposes that

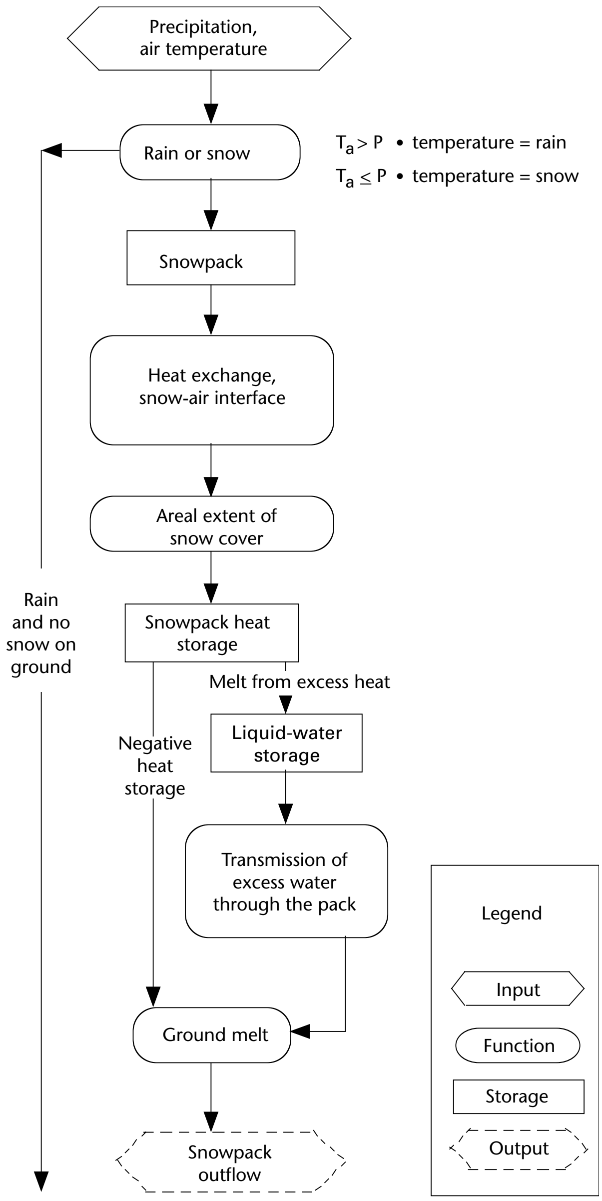


Figure II.6.14. Snow accumulation and ablation model flow chart

the historic sequences are representative of what can be expected in future years.

6.3.4.4 Input data

Input data for use in physically based or index types of conceptual models may be either precipitation measurements and/or measurements of the water equivalent of the snow cover. With physically based conceptual models, corrections should be made for systematic errors (see Volume I, 3.3.6) in the precipitation measurements, so that the input data are as representative as possible of the average precipitation and/or snowcover. In mountainous regions, where the snowcover is highly dependent on altitude, the observations from meteorological stations are often affected by local exposure including wind, local slope and aspect and must be adjusted to better represent the average meteorological conditions if they are used to simulate the snowcover conditions. In practice, snowcover and precipitation measurements complement each other.

The spatial distribution of snowcover is often best described by snowcover depletion curves which summarize the per-cent areal coverage of the snowpack as it increases in average depth. Watershed-wide snowcover depletion curve relationships are currently used in lumped hydrological models such as the National Weather Service River Forecast System, or NWSRFS (Anderson, 1973), to describe the snowcover distribution as the snowcover melts. These relationships are difficult to obtain and require calibration for each specific watershed. The simplest representation of snowcover is uniform snowcover, which is of constant depth and complete areal coverage. Knowledge of the areal distribution of the snowcover within and between land units is required to make reasonable estimates of the total water available in the snowcover of a watershed. The areal distribution of the snowcover within a land-unit type can then be summarized in the form of an areal distribution curve. An areal distribution curve is a summary of the state of the snowcover at a given time within a basin. Intense sampling programmes are required to develop datasets to quantify the snow distribution in the form of areal distribution curves.

Since it is not practical to physically model the distribution of snowcover, the development of statistical or empirical distribution relationships based on landcover and physiographic considerations is a sensible approach to the problem. This is accomplished by the use of snowcover depletion curves. Rango and his colleagues present a depletion curve where the percentage of snow-covered area is on the y-axis and time is on the x-axis, providing a conceptually based approach to understanding basin snowmelt (Rango and others, 1983).

6.3.4.5 Theory of snowmelt at a point

A rational approach to estimating the rate of snowmelt is based on an energy budget, which accounts for the significant modes of heat exchange. Heat is transmitted to snow by absorbing solar radiation, net long-wave radiation, convective heat transfer from the air, latent heat of vaporization by condensation from the air, relatively small amounts of heat from rain and generally negligible amounts of heat from the underlying ground.

The equation for energy balance can be used to determine the amount of energy available for snowmelt, Q_m, which can be directly transformed into the amount of snowmelt for a unit cube of snow:

$$Q_m = Q_n + Q_h + Q_e + Q_g + Q_a - dS_i/dt \quad (6.38)$$

where energy fluxes (per unit area) are respectively: Q_n net longwave radiation, Q_h sensible heat transfer due to temperature difference between the surface and the air, Q_e latent energy flux caused by water vapour change (release of heat by condensation or its removal by sublimation or evaporation), Q_g conduction of heat from the underlying ground, Q_a advection of heat (rain), and S_i snowpack heat storage.

A melting snow cover typically contains from two to five per cent by weight of liquid water, but occasionally as much as 10 per cent is held for brief periods when melting rates exceed transmission capacity. Thus, for short periods of time, the total release of water from a snow cover may slightly exceed the amount of snow actually melted by the prevailing meteorological conditions. For practical purposes, this release of previously melted water is implicitly incorporated into the empirical constants, which are therefore burdened with uncertainties.

Absorbed solar radiation varies with latitude, season, time of day, atmospheric conditions, forest cover, slope, orientation of surface and the reflectivity of the snow. The effects of latitude, season, time of day and atmospheric conditions are included in solar radiation observations, which must generally be interpolated because of the sparse network of such stations. These effects may also be computed

on a daily total basis by means of formulae or diagrams that express solar radiation as a function of degree of cloudiness, time of year and latitude.

The effect of forest cover on the transmission of solar radiation is important, and in experimental areas, it has been expressed as an empirical factor that relates the transmission coefficient to canopy density. Usually direction and steepness of slope and forest cover are represented by constant factors, derived empirically for a given drainage area.

Reflectivity of a snow surface ranges from about 90 per cent for newly fallen snow to about 40 per cent for old snow that is coarse grained and which is ordinarily covered late in the season by a thin layer of dark debris such as wind-blown organic or mineral dust. In middle latitudes during late spring, an unforested snow cover with low reflectivity commonly absorbs sufficient solar radiation to melt 50 millimetres of water equivalent per day.

Long-wave radiational exchange is the difference between outgoing radiation from the snow surface and downward radiation from clouds, trees and the atmosphere. With dense low clouds or heavy forest cover warmer than 0°C, the exchange is a gain to the snow. Long-wave radiation from the atmosphere in the absence of clouds or forest cover is largely a function of air temperature and is nearly always less than the loss from the snow. Long-wave radiational exchange commonly ranges from a gain of heat equivalent to as much as 20 mm of melt water per day to a loss equivalent to 20 mm per day.

The main factors in the convectional exchange of sensible heat are the temperature gradient in the air immediately above the snow and the intensity of turbulent mixing expressed by horizontal wind speed.

The principal factors in heat from condensation are the vapour-pressure gradient and intensity of turbulent mixing, which may be indicated by wind speed. The combined exchange of sensible and latent heat by turbulent exchange may range from a gain of heat that is equivalent to more than 100 mm of melt per day to a loss corresponding to two or three millimetres. The potential gain greatly exceeds the potential loss because the temperature and vapour-pressure gradients for heat gain can be very great with the snow temperature limited to 0°C, whereas with very low air temperatures and vapour pressures accompanying the loss of heat, the snow-surface temperature generally falls correspondingly. Thus, the gradients are reduced. Heat gain from warm rain can be computed from the latent heat of fusion of the ice (80 calg^{-1}) which comprises the snow, and the temperature of the rain, which can usually be taken as the wet-bulb temperature of the air. Computations show that an unusually heavy rain – at least 120 mm of rain with a temperature of 16°C – is required to produce as much as 25 mm of snowmelt in a day.

The conduction rate of heat from the soil to a newly formed snow cover may be rapid for a short time, but the usual geological gradient of temperature and the gradient of temperature after steady-state has been established produce less than about one millimetre of snowmelt per day.

The foregoing rates of snowmelt from various modes of heat exchange are not additive. For example, the conditions for maximum turbulent exchange would occur during stormy weather and not with maximum solar radiation. Numerous equations have been published expressing the modes of heat exchange in terms of observable elements. For further information, please refer to WMO-No. 749, Operational Hydrology Report No. 35 – *Snow Cover Measurements and Areal Assessment of Precipitation and Soil Moisture* (WMO, 1992) and WMO-No. 646, Operational Hydrology Report No. 23 (WMO, 1986) – *Intercomparison of Models of Snowmelt Runoff*.

The integration of a rational snowmelt function over a heterogeneous drainage area of significant size is extremely difficult at best and practically futile without elaborate instrumentation. Estimating the quantity or rate of melt is based on water budget accounting in addition to heat budget accounting. In the absence of rain, radiational exchange is relatively important, and consequently the effects of snow reflectivity and forest canopy density are important; however, these are rarely measured. During periods of heavy rain, the rate and amount of snowmelt may be no greater than the error in estimating the amount and effects of the rain. During storms accompanied by considerable turbulent mixing and heavy, low clouds, there is relatively little short-wave solar radiation, and long-wave radiation, convection and condensation are the major sources of heat. The difficulty of separating the contribution of rain from that of snowmelt has left the question of snowmelt during rain largely in the realm of theory with very little empirical evaluation (US Army Corps of Engineers, 1960). Daily solar radiation for a given latitude and time of year is influenced by local cloudiness, which in turn is observed subjectively and sparsely – rarely with

respect to its radiative transmissivity. Further, there is the problem of determining the active or contributing area of the snow.

The active or contributing area may be defined as the area over which snow is melting or over which snowmelt reaches the soil. This area, however defined, varies diurnally. If the diurnal cycle includes nocturnal freezing, some account must be taken of the heat and moisture storage involved. Early in the melting period, some heat is necessary to raise the temperature of the snow to 0°C and to melt sufficient snow to meet the water-holding capacity of the snow cover. This heat is relatively small with respect to the total heat required to melt the snow cover.

The most widely applied method for estimating basin-wide snowmelt is the use of degree-day factors. Temperature data are usually available, and the variation of temperature over a drainage area can generally be determined for deriving and applying degree-day functions. The rationale for the degree-day method is twofold. First, air temperature near the snow is largely a physical integration of the same modes of heat exchange that melt snow. Second, each mode of heat exchange can be related to air temperature except during abnormal winds. For example, minimum daily air temperature is highly correlated with dewpoint temperature, which determines the vapour-pressure gradient for condensation melting. Maximum daily temperature or temperature range is an index of solar radiation. Within its usual range, long-wave radiation can be expressed as a linear function of air temperature.

Efforts have been made to give the maximum and minimum daily temperatures various weights and to use degree-day bases other than 0°C. Efforts have also been made to divide the day into smaller time units and to use degree-hour factors. However, the diurnal cycle of heat exchange and snowmelt makes the day a logical and convenient unit for snowmelt, and the usual degree-day base is 0°C, which is generally taken as the mean of the daily maximum and minimum air temperatures. Point snowmelt degree-day factors for several mountainous regions in the middle latitudes of North America have been averaged in Table II.6.3, in millimetres of melt, and the mean of daily maximum and minimum temperature above a base of 0°C. Individual values may depart widely from these averages.

Similar degree–day factors are given in Table II.6.4 for lowlands in moderate latitudes of the former Union of Soviet Socialist Republics.

With a shallow snow cover, the storage and delay of melt water passing through the cover are generally inconsequential, compared with storage and delay in the soil mantle and uncertainties in the amount of snowmelt itself. The time required for liquid water to drain from a snow cover is about one hour, plus an hour for each 50 cm of depth.

Areal variations in melting rate and in the distribution and diminishing size of the area covered by snow during a melting period are related to fairly permanent characteristics of the catchment area, such as its topography and distribution of vegetative cover. Consequently, the melting rate over a catchment reflects a fairly consistent trend in contributing area and snow condition during a melting period. This trend influences the shape of empirically defined S-shaped curves such as those in Figure II.6.15. Because of the areal dispersion of the snow and of its local melting rates, some of the snow starts to melt before the rest. Thus, the average melt rate per unit area is low early in the melting period and increases as more of the area contributes. Toward the end of the melting period, the slopes of the curves of Figure II.6.15 diminish because of the diminishing area of snowmelt contribution. The steepest portions of the curves occur after melting conditions have become established over a large contributing area. The proportionality of melting rates to the initial

Table II.6.3. Degree–day factors (mm °C^{-1}) for mountainous regions in North America

Month	*Moderately forested*	*Partly forested*	*Non-forested*
April	2	3	4
May	3	4	6
June	4	6	7

Table II.6.4. Degree–day factors for lowland regions in the former Union of Soviet Socialist Republics

Area	*Degree–day factors (mm °C^{-1})*
Non-forested areas	5
Sparse coniferous and average density of harwoods	3–4
Average density of coniferous woods and dense mixed woods	1.7–1.8
Dense coniferous woods	1.4–1.5

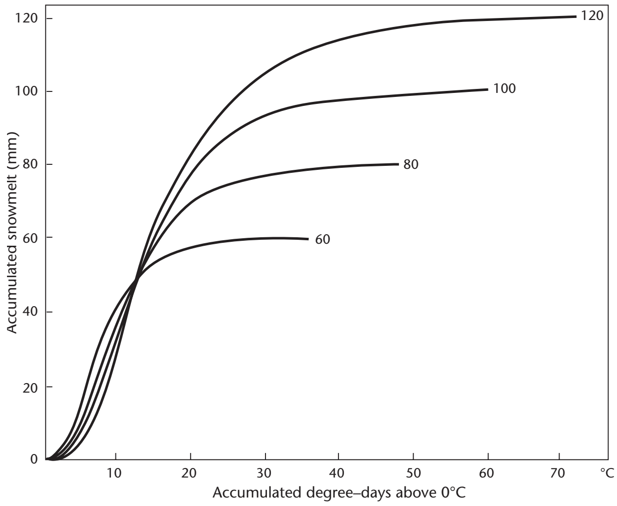


Figure II.6.15. Typical degree–day snowmelt relationship for a catchment for different values of initial water equivalent

quantities of snow comes largely from the fact that, with more snow, the contributing area is larger. The steepest portions of the curves of Figure II.6.15 have a slope that corresponds to the values of Tables II.6.3 and II.6.4.

Evaporation loss from the snow cover is negligibly small during brief melting periods and may be more than balanced by condensation on the snow surface. Equations for condensation on a snow cover may also be used to estimate evaporation from the snow. Measuring evaporation from a snow or ice surface is difficult and probably about as accurate as the computation of evaporation in general. It is estimated that during winter periods, evaporation occurs from a typical snow surface at rates ranging from zero to as much as 20 mm per month. During melting periods, condensation tends to prevail and occurs at rates spanning from zero to possibly as much as 10 mm of condensate per day.

In mountainous regions, where great quantities of snow accumulate, where the melting season may cover several months and where melting conditions vary greatly with a large range of elevation, the reliability of curves such as those of Figure II.6.15 are limited. Evaporation during long warm periods may be significant. During the melting season, successive aerial or other surveys show the changing snow-covered area and meteorological observations are interpreted to express the variation of melting rate with elevation. The contribution of snowmelt should be determined by elevation zones. In addition, with deep mountain snow cover, more consideration must be given to the retention of melted snow in the cover.

Higher than average degree–day factors should be used when unusually high wind speeds or humidity occur.

6.3.4.6 Estimating snowmelt inflow rates

To determine the total snowmelt runoff in lowland basins, water balance studies can be adopted. From these, the expected total snowmelt runoff can be estimated at the beginning of the snowmelt period. However, values of the daily snowmelt inflow are often required for hydrograph calculations. The following factors should be taken into account when estimating these values:
(a) Heat inflow to the snow cover;
(b) Water-retention capacity of the snow cover;
(c) Area covered with snow;
(d) Water-retention capacity of the basin.

6.3.4.7 Probable maximum precipitation and snowmelt

In the case of very large basins at high latitudes, snowmelt, rather than rainfall, may be the primary cause of the probable maximum flood. Flood-runoff volume and temporal distribution are then based on the estimation of snowmelt resulting from the estimated maximum values of temperature, wind, dewpoint and insolation in a manner analogous to maximization of storm rainfall.

A more common situation in lower latitudes is for rainfall to be the primary factor producing the probable maximum flood with snowmelt adding an increment to the maximum hydrograph. Snowmelt, compatible with estimated synoptic conditions accompanying the maximized storm, is then added to the maximized rainfall depth.

For some basins, only a detailed analysis will reveal whether the probable maximum flood will result from a cool-season rainstorm combined with snowmelt or from a summer rainfall that may be more intense but cannot logically be expected to occur in combination with snowmelt.

6.3.4.7.1 *Probable maximum snow accumulation*

The snowmelt contribution to the probable maximum flood will depend on the maximum rate of melting and the water equivalent of the snow cover available for melting. Water equivalent of a snow cover is the depth of water that would result from

melting and depends on the snow density as well as its depth. Various methods have been used to estimate probable maximum snow accumulation; the three most common are as follows:

(a) Partial-season method – The highest observed snow accumulations in each month or two-week period, according to the frequency of observations, are combined, regardless of the year of occurrence of each observation, to give a synthetic year of very high snowfall. The method can be applied to shorter time intervals, such as a week or four-day period, if suitable records are available;
(b) Snowstorm maximization – The ratio of maximum atmospheric moisture content in the project area at the time of year at which a snowstorm occurs to the actual moisture content of the snowstorm is determined. The observed snowfall produced by the snowstorm is multiplied by this ratio to give maximized snowfall for the snowstorm. Maximization of moisture content must be restricted to a value that will produce snow and not rain;
(c) Statistical methods – A frequency analysis of precipitation and snow-depth records is made to determine the values for various return periods. Analyses are made of three types of data: station precipitation depth, basin snowfall depth and water equivalent of snow on the ground.

6.3.4.7.2 *Snowmelt estimation*

Owing to the complex spatial and temporal variability of snowmelt over most catchments caused by differences in slope, aspect, forest cover and depth of snow cover, the degree–day method is often adopted as a practical solution to the problem of estimating snowmelt over a catchment. Maximum degree–day conditions may be estimated from temperature records for the project basin or a neighbouring area and may be applied to the estimate of probable maximum snow accumulation to provide an estimate of probable maximum flood runoff.

For probable maximum conditions, the air temperature and wind speed are made consistent with the assumed synoptic conditions accompanying the storm-producing probable maximum rainfall. It also is assumed that an optimum snow cover exists. Optimum in this situation means the following:

(a) The snow cover has only sufficient water equivalent to melt completely during the storm;
(b) The snow cover has been melting and contains a maximum amount of liquid water;
(c) The water equivalent of the snow cover is distributed so as to be at a maximum where the melting is maximum, which is different from the usual situation of increasing the snow-cover water equivalent with increasing elevation.

6.3.4.8 Runoff from short-period snowmelt

In a plains region, where increments of runoff are relatively small and the melting period is brief, runoff may be estimated by incorporating estimated snowmelt obtained by methods such as described above, into a rainfall–runoff relationship (see 6.3.2). It may be necessary to use the relationship in a way that reflects a high percentage of runoff because the snow cover or cold weather inhibits evapotranspiration losses antecedent to the melting period. In mountain catchment areas, where deep snow covers prevail and the melting season lasts several months, methods commonly used for estimating runoff from brief rainstorms do not necessarily apply. Runoff from the melt that occurs on a particular day is ordinarily spread over a long period, overlapping the melting increments of many other days. Also, evapotranspiration losses, which may be neglected during a period of rainfall, become important during a long melting season. One way to estimate runoff from day-to-day snowmelt is first to estimate the seasonal volume of runoff and then to distribute it in accordance with observed or estimated local daily melting rates (6.3.4.6 and 6.3.4.7), basin-storage characteristics, contributing area and seasonal evapotranspiration. Basin storage and lag may be accommodated by routing though an analogous system of reservoirs with constants determined empirically from historical basin data. Where the catchment is so small that the diurnal increments of snowmelt are not damped out by storage, six-hour – rather than daily melting increments should be used, or a characteristic diurnal distribution can be introduced into the routing method.

6.3.4.9 Snowmelt runoff analysis using remote-sensing

Snowmelt runoff procedures have followed two distinct paths: an empirical approach and a deterministic modelling approach. The choice of approach depends on both the availability of data to quantify the snowpack and the extent of detail required of the output. In order to make accurate estimates of snowmelt runoff, hydrologists need to quantify the snowmelt in the following terms: the areal extent of the snow, S; the snow water equivalent, SWE; and the condition or properties of the snow such as depth, density, grain size and

presence of liquid water (Engman and Gurney, 1991). The gradually decreasing areal extent is a characteristic feature of the seasonal snow cover. Regardless of the approach used to conduct day-to-day simulations of snowmelt runoff, whether it be an empirical approach based on historical data or a deterministic approach, it is sufficient to know the daily snow-covered area in the basin without knowing the initial accumulation of snow in terms of water equivalent (WMO, 1994).

For many basins, there is a very good relationship between runoff and snow cover area (Engman and Gurney, 1991). For operational runoff forecasts, however, the water equivalent must also be determined (WMO, 1994). Remote-sensing offers a new valuable tool for obtaining snow data in order to predict snowmelt runoff (Engman and Gurney, 1991). Ostrem and others (1991) developed a method using data from the National Oceanic and Atmospheric Administration (NOAA) and the television infrared observation satellite (TIROS) to measure the remaining snow and predict the corresponding snowmelt runoff volume for a number of Norwegian high mountain basins. Many large hydropower companies use snow cover extent maps from NOAA AVHRR, advanced very high-resolution radiometer, on an operational basis as input to their hydropower production planning (Andersen, 1991).

Remote-sensing techniques using appropriate wavelength bands allow to a certain degree the estimation of snowcover features such as grain size, albedo, layering, surface temperature and snowpack temperature. This, in turn, allows a good estimate of the time when the snowpack is ready to transmit melt water from the surface to lower layers, known as a ripe condition, and to eventually produce runoff at the base of the snowpack (Rango, 1993). The first empirical approach to snowmelt runoff estimation using remote-sensing was developed by Rango and others (1977); they used satellite-observed snow cover data in empirical regression models developed for the Indus and Kabul Rivers in the Himalayas. Martinec and Rango (1987) and Rango and van Katwijk (1990) later used remotely sensed snow-water-equivalent and temperature data to construct modified snow cover depletion curves for use in the snowmelt-runoff model for snowmelt forecasts in the Rio Grande basin.

Overall, remote-sensing is very successful in mountain regions, especially when the aim is to map snow cover. This is hindered only in regions with very dense forest cover.

New models developed to use remote-sensing data will also improve snow hydrology predictions. Further, the merging of remote-sensing data with digital elevation modelling and geographical information systems enables different types of data to be combined objectively and systematically (Engman and Gurney, 1991). Digital elevation modelling is used to normalize imagery by using the elevation of the sun and the slope, aspect and elevation of the terrain (Baumgartner, 1988; Miller and others, 1982). Geographical information systems are helpful in combining vegetation masks with satellite imagery (Keller, 1987).

6.3.5 Streamflow routing

Runoff from a headwater area moves downstream as a wave whose changing configuration at various stations can be computed by a technique known as flood routing. Storage and other effects tend to attenuate the wave. Irregularities in channel conditions and tributary inflows are inherent complexities of the problem. The routing of flood waves through reservoirs and channels is accomplished by many methods.

6.3.5.1 Hydrodynamic methods

Hydrological research has gained much knowledge of the physical processes that comprise the water cycle in nature. Similarly, the high technology employed in continuous data acquisition and integration in time and space, combined with modern computers, permit rapid processing of hydrological and meteorological data of all types. All this has helped improve the third type of modelling, hydrodynamic modelling.

Hydrodynamic models are based on numerical integration of the equations of momentum and mass conservation that describe the physical processes in the basin. Since hydrodynamic models are based on the physical laws governing the processes, extrapolation beyond the range of calibration may be performed more confidently than with conceptual models. Complete dynamic routing, which accounts for flow-acceleration effects and the water-surface slope, can determine flows and water-surface elevations accurately in the following unsteady flow situations:

(a) Upstream movement of waves, such as those produced by tidal action or sea-storm surges;
(b) Backwater effects produced by downstream reservoirs or tributary inflows;
(c) Flood waves occurring in rivers having flat bottom slopes: less than 0.05 per cent;
(d) Abrupt waves caused by controlled reservoir releases or by the catastrophic failure of a dam.

Dynamic routing is generally based on the one-dimensional hydrodynamic equations of unsteady flow, known as the Saint Venant equations. These equations are generally expressed in their conservative form below.

Continuity:

$$\frac{\partial Q}{\partial x} + \frac{\partial s_c (A + A_0)}{\partial t} - q = 0 \tag{6.39}$$

Momentum:

$$\frac{\partial (s_m Q)}{\partial t} + \frac{\partial (\beta Q^2/A)}{\partial x} + gA\left(\frac{\partial h}{\partial t} + S_f + S_{ec}\right) - q v_x + W_f B = 0 \tag{6.40}$$

in which:

$$S_f = \frac{n^e Q}{A^2 R^{4/3}} \tag{6.41}$$

where Q is discharge, A is the active cross-sectional area, A_0 is the inactive or dead-storage cross-sectional area, s_m is a depth-weighed sinuosity coefficient, S_{ec} is the expansion-contraction slope, β is the momentum coefficient for non-uniform velocity distribution within the cross-section, $W_f B$ is the resistance effect of wind on the water surface, h is the water-surface elevation, v_x is the velocity of lateral inflow in the x-direction of the river, B is the top width of the active cross-sectional area, n is the Manning roughness coefficient, R is the hydraulic radius; other symbols are as previously defined, except that:

$$S_{ec} = \frac{K_{ec} \Delta (Q/A)^2}{2 g \Delta x} \tag{6.42}$$

where K_{ec} is the expansion and contraction coefficient, $\Delta(Q/A)^2$ represents the difference in the term $(Q/A)^2$ at two adjacent cross-sections separated by a distance Δx.

No analytical solutions of the complete non-linear set of equations 6.39 to 6.41 exist. The numerical techniques for solving the aforementioned equations for natural rivers may be classified into two broad categories: the method of characteristics, which is not widely used nowadays, and finite-difference methods in explicit and implicit schemes, which are very common. Finite-difference methods transform partial differential equations 6.39 and 6.40 into a set of algebraic equations. The explicit methods solve these algebraic equations sequentially, at each cross-section, computational reach, and at a given time, while the implicit methods solve algebraic equations simultaneously for all computational reaches at a given time.

There are advantages and disadvantages to the various solution techniques. Factors such as numerical stability and convergence, required computational time and computer storage, and degree of programming and mathematical complexity must be considered. Some solution techniques require modifications to the form of equations 6.39 and 6.40 before they can be applied.

In general, implicit finite difference techniques are more complex but more efficient than explicit methods when calculating unsteady flows of several days duration. Much larger time steps can be used with the implicit techniques. Explicit techniques are simple; they are confronted, however, with numerical stability problems unless the time step is properly selected. These and other limitations should be thoroughly understood before selecting a particular solution technique to develop a dynamic-routing forecasting method or select an existing dynamic-routing technique for a particular application.

A critical task in applying dynamic routing to an actual forecast situation is the determination of the roughness parameter in the S_f, friction-slope term given by equation 6.40. The roughness parameter often varies with flow or elevation, as well as with distance along the river. A prior determination of the roughness parameter relationship with flow and distance by trial and error is very time consuming. Techniques for automatically determining the relationship greatly facilitate the operational utilization of dynamic routing in a forecasting environment. A proper evaluation of the boundary and the initial conditions for solution of the Saint Venant equations in an operational mode is another critical task in the implementation of dynamic routing techniques.

Another critical task is the establishment of an efficient data-acquisition and management programme linked integrally with the computational element. Cross-section geometry should be processed as efficiently as possible for use by the dynamic-routing program. Anticipated flow conditions should require as little data entry as possible for a dynamic-routing technique to be feasible for use as an operational forecasting tool.

By slightly rewriting the momentum equation and ignoring momentum from lateral inflows, a very clear picture can be obtained showing the

fundamental differences between dynamic, diffusion, and kinematic routing.

Consider:

$$\frac{1}{g}\frac{\partial h}{\partial t} + \frac{v}{g}\frac{\partial v}{\partial x} + \frac{\partial h}{\partial x} - s_o + s_f = 0 \qquad (6.43)$$

kinematic model
diffusion model
dynamic model

At the first level of approximation, the terms representing the accelerations related to the time variation of inflow and the spatial variation in velocity are neglected. The resulting model is referred to as the diffusion model. In some flow situations, it is also possible to neglect the pressure-force term and treat the momentum equation as a balance between the forces of gravity and friction. This approximation is known as the kinematic model.

Both the kinematic and diffusion approximations have been used successfully to describe overland flows and flows in streams where slopes are greater than approximately 0.1 per cent. The diffusion model can be used on rivers with smaller slopes, but with caution because the inertia terms may become important. The kinematic model has become popular in applications where the irregular geometry and topography of natural catchments can be replaced by a series of simple elements, such as flow planes and regular channel segments. The kinematic equations are also used in water quality models that predict the transport of pollutants. A kinematic model does not consider backwater effects from lateral inflows or downstream reservoir operations, nor can it be used to predict wave progressions in the upstream direction.

6.3.5.2 Hydrological methods

Hydrological flood-routing methods use only the continuity equation, or mass conservation law. In these techniques, only the wave propagation is studied by considering the increases and decreases of storage in a reach lying between two measuring points. However, because the relationship between storage and flow is determined empirically by these methods, they cannot be used directly when flow data or levels are required for design purposes.

When hydrological routing methods are employed, the flow at an upstream point is given or assumed, and routing is used to compute the flow and stage at a downstream point. Routing consists of the solution to the following continuity equation by using a relationship between storage and flow:

$$I - Q = dS/dt \qquad (6.44)$$

where I and Q are the discharges at upstream and downstream points, respectively, S is storage in the river reach between the upstream and the downstream cross-sections, and t is time. Solution of this equation involves approximations concerning the storage–flow relationship, the main difficulty in hydrological streamflow routing. However, with sufficient hydrometric data, this relationship can be derived empirically.

The simplest routing methods are based on linear storage–flow relationships, which make it possible to obtain analytical solutions. Two such methods are applicable in short-range forecasting practice, as indicated below.

(a) The Muskingum method, which is based on the following storage–flow relationship:

$$S = K\,[xQ_1 + (1 - x)Q_2] \qquad (6.45)$$

The constants K and x are derived empirically for a given reach from discharge data. They can be determined by plotting S versus $xI + (1 - x)Q$ for various values of x. The best value of x is that which results in the data plotting most closely to a single value curve.

The Muskingum method is often used in the following discrete form:

$$Q_{j+1} = C_1\, I_{j+1} + C_2\, I_j + C_3\, Q_j \qquad (6.46)$$

where C_1, C_2 and C_3, being functions of Muskingum parameters K and x and the time step Δt, sum to unity ensuring that the sum of the constants is equal to unity;

(b) The specific reach method, proposed by Kalinin and Miljukov (1958), is based on the following linear storage–flow relationship:

$$Q = K\,S \qquad (6.47)$$

where K is the storage constant equal to the travel time through the reach. The above equation is applicable to transit reaches of specific length, L, which is roughly equal to:

$$L = \frac{Q}{Z\,\dfrac{\partial Q}{\partial h}} \qquad (6.48)$$

where Z is the slope of the water surface, and $\partial Q/\partial h$ is the tangent of a stage–discharge relationship. If a river segment consists of several specific reaches, routing is carried out in succession from one specific reach to the next downstream. The computed discharge for the downstream point of the first reach is taken as the inflow for the second reach, and so on.

The following formula expressing the transformation of flow by a system of identical linear reservoirs can be used for long river reaches that lack the data needed to determine the number of specific reaches:

$$Q(t) = I_0 \frac{\Delta t}{K^N (N-1)!} t^{N-1} e^{-t/K} \tag{6.49}$$

where N is the number of characteristic reaches or reservoirs, K is the travel time for one characteristic reach, I_0 is the inflow into the first characteristic reach and t is the time. The K and N parameters are determined by trial and error or optimization.

6.3.5.3 Reservoir routing

A reservoir leads to a decrease in the peak discharge, compared with that which would have occurred had the reservoir not been in place because the passage of a flood through a reservoir differs somewhat from its passage through a channel.

Because the velocity of the flood wave in a reservoir is higher than in channels, the delay in the peak outflow with respect to the peak inflow does not necessarily mean a delay with respect to the peak that would have occurred under the conditions prevailing prior to the construction of the reservoir. Furthermore, the construction of a reservoir may sometimes worsen downstream flood conditions, despite its effect in decreasing peak discharges. The attenuated peak may occur in phase with peaks of tributaries that are usually out of phase. Thus, it should not be taken for granted that reservoir construction will improve downstream flood conditions. The hydrology and the hydraulics that would prevail under the design conditions should be studied carefully.

6.3.5.4 Dam breaks

Catastrophic flash flooding results when a dam fails, and the outflow, through the breach in the dam, inundates the downstream valley. A dam that fails can be man-made or, for example, ice jam or flow debris. Often the dam-break outflow is several times greater than any previous flood on the river concerned. Little is known of failure modes of artificial or natural dams. Hence, real-time forecasting of dam-break floods is almost always limited to occasions when failure of the dam has actually been observed. Different failure modes may be assumed for planning calculations when the implications to downstream development are investigated with regard to zoning or evacuation contingency plans.

Earlier classical studies of this problem have assumed instantaneous dam failure and idealized downstream conditions. More recently, engineers have sought to approach the problem by assuming a triangular-shaped outflow hydrograph based on the Schocklitsch or similar maximum-flow equation:

$$Q_m = \frac{8}{27} W_d \sqrt{g Y_0^3} \tag{6.50}$$

where g is the acceleration due to gravity, W_d is the width of the breach and Y_0 is the height of the water behind the dam. By using equation 6.49 and an empirical recession coefficient, the synthesized hydrograph is routed through the downstream valley via a hydrological-routing technique. Alternatively, a more realistic approach can be found in dynamic-routing techniques (see 6.3.4.2) to route the rapidly changing and relatively large dam-break flood wave. Explicit account is taken of downstream dams, overbank storage, downstream highway embankments, and expansion and contraction losses.

As time is essential in real-time forecasting of a dam-break flood, operational techniques must be computationally efficient. However, an even more important consideration is the data requirement of the forecast technique. If dynamic routing is to be used, every effort should be made to minimize the amount of cross-sectional data needed in the routing phase of the forecast, and all data and program files must be immediately available for use.

6.3.6 Modelling other processes

6.3.6.1 Sediment transport modelling

Sediment transport models predict sediment transport rate and direction based on water surface elevations or velocities determined by using a hydrodynamic model (see 6.3.4.2), an essential part of the sediment transport model based on the numerical solution of the Saint Venant equations of continuity and momentum.

Basic processes in sediment transport can be broken down into erosion, entrainment, transportation and deposition. Sediment on the streambed will remain immobile only as long as the energy forces in the flow field remain less than the critical shear stress threshold for erosion. After the critical shear stress is reached, the sediments begin moving by jumping or bouncing, rolling and sliding. This movement is known as bed load. Various researchers have developed relationships describing this bed load as a function of the bed shear stress and the grain size diameter. These are known as sediment transport functions and are mostly applicable to non-cohesive material (see 4.8.6).

Computation of the particle-settling velocity is necessary for several non-cohesive sediment transport functions.

$$\omega_f = F\sqrt{dg\,(G-1)} \tag{6.51}$$

where:

$$F = \left[\frac{2}{3} + \frac{36\,v^2}{g\,d^3(G-1)}\right]^{1/2} - \left[\frac{36\,v^2}{g\,d^3(G-1)}\right]^{1/2} \tag{6.52}$$

for particles with diameter d between 0.0625 mm and 1 mm. For particles greater than 1 mm, $F =$ 0.79. In the above equations, ω_f is the fall velocity of sediments, g is the acceleration due to gravity, G is the specific gravity of the sediments and v is the kinematic viscosity of water.

Most of the sediment transport models allow use of more than one function, since there is no universal function that can be applied accurately to all sediment and flow conditions. Most of these transport functions were developed to compute total bed load without breaking down the load by size fraction. Some transport models apply these functions for different size fractions to account for variation in the bed load grain size distribution and can simulate bed-material mixing processes, and therefore armouring effects.

The bulk of sediment in transport can be characterized as being transported in suspension. Suspended load calculations include the time-space lag in the sediment transport response to changes in local hydraulic conditions. Cohesive sediments in transport will remain in suspension as long as the bed shear stress exceeds the critical value for deposition. Cohesive sediments tend to segregate to low density units, a process that is strongly dependent on the type of sediment, the concentration of ions in water and flow conditions, and the settling velocity that is no longer a function of particle size. This aggregation is accounted for in the models by assigning settling velocities. In general, simultaneous deposition and erosion of cohesive sediments do not occur, but the structure of cohesive sediment beds does change with time and with overburden.

There may be no net change in the elevation of the bed unless the erosion rate is different from the deposition rate; these are two processes that go on continuously and independently. The change of bed level may be determined by a sediment continuity equation. The equation is derived based on the assumption that the changes in volume of suspended sediment are much smaller than the changes in bed sediment volume, which is generally true for long-term steady-flow simulations. The mass conservation equation for sediment reduces to:

$$\frac{\partial Q_s}{\partial x} + \varepsilon\,\frac{\partial A_d}{\partial t} - q_s = 0 \tag{6.53}$$

where ε is the volume of sediment in a unit bed layer volume (one minus porosity), A_d is the bed sediment volume per unit length, Q_s is the volumetric sediment discharge and q_s is the lateral sediment inflow per unit length.

Certain sediment transport and morphological models, such as MIKE 21C, consider helical flows in connection with sediment transport simulations in order to simulate the development of bend scour, confluence scour and the formation of point bars as well as alternating bars. These models do provide curvilinear computational grids which are more suitable for river morphology modelling. Bank erosion is included at each computational time step. The eroded bank material is included in the solution of the sediment continuity equation. Bank erosion will produce a retreating bank line, which is modelled by the movement of the adaptive curvilinear grid.

Additional information on sediment modelling is to be found in 4.8.6.

6.3.6.2 Water quality modelling

The management of water quality in natural and artificial water bodies is a complex task that requires monitoring of the water quality characteristics, interpretation of the monitored data in relation to causative factors and prediction of future changes of these characteristics in terms of the various

management alternatives under consideration. The solution to these problems can be greatly aided by the use of water quality models. These enable prediction on the basis of the following factors:
(a) A series of input data on pollution inflow;
(b) Meteorological-environmental initial conditions;
(c) Hydraulic-hydrological and land-use characteristics of the water body and its watershed;
(d) The evolution in time and/or space of certain water quality characteristics of the water body considered for various water management alternatives.

Water quality models are frequently linked to hydraulic and hydrological models.

Mathematical water quality models may be classified according to the general taxonomy of models (see 6.1) and according to the following criteria:
(a) Water quality constituents: into single- or multi-constituent models;
(b) Type of constituent modelled: into conservative, for example, salt; non-conservative physical, for example, temperature; non-conservative chemical, for example, dissolved oxygen; or non-conservative biological, for example, coliform bacteria.

For the description of pollutant transport in rivers, the most commonly used model in practical applications is the one-dimensional model based on the advection-dispersion equation:

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = D_L\frac{\partial^2 c}{\partial x^2} \qquad (6.54)$$

where c is the pollutant concentration, u is the mean water velocity, D is the longitudinal dispersion coefficient, t is time and x is distance.

The longitudinal dispersion coefficient is calculated on the basis of the Fisher equation:

$$D_L = \frac{0.07u'^2 l^2}{\varepsilon_z} \qquad (6.55)$$

in which u'^2 is the deviation from the cross-sectional mean, l is the distance from the thread of the maximum velocity to the most distant bank and ε_z is the transverse mixing coefficient.

To apply this model to pollutant transport in a river, the river is divided into reaches, each several kilometres in length, within which the water velocity is considered to be constant. The water velocity within each sector is calculated by means of a hydraulic or hydrological model (see 6.3.4).

Water quality models can be used in water quality management for several purposes, including the design of water quality monitoring networks in space and time, the interpretation of data obtained in relation to factors determining water quality, interfacing with other environmental (air, soil) pollution models and with ecological models, the assessment of trends in water quality with or without various alternative pollution-correction measures and the forecast of the arrival time of a pollutant and of a concentration profile along the river.

Water quality models have been applied with various degrees of success to the solution of water quality management problems in many countries (Biswas, 1981). For example, a relatively simple model was used for investigating the effect on water quality of large-scale water transfers from the river Severn into the river Thames in the United Kingdom of Great Britain and Northern Ireland. The model was used to assess the effect of such transfers on the concentration of a number of conservative and nearly conservative substances contained in the water. The model was based on river-flow separation according to source, for example, surface, interface and base flow, and on developing relationships between the concentrations of the determinants considered and the water inflow and inflow variation for each source. The simulation results matched the recorded data reasonably well.

Another example of the practical application of a water quality model for water management purposes is the study of the effect of removal of biochemical demand loads by waste-treatment plants on the dissolved oxygen concentration in the water of the Thames river in Ontario, Canada. The results indicate that obtaining dissolved oxygen concentrations above the criterion accepted for good water quality by removing biochemical demand loads is feasible at one point, while at another point, this would be very difficult. *Hydrological Aspects of Accidental Pollution of Water Bodies* (WMO-No. 754) provides a detailed review of a number of water quality models applied in Canada, France, Germany, Poland, the United Kingdom and the United States to a variety of rivers having significant pollution problems.

Water quality models are also used for computing the propagation of accidental pollution events. Such models have been operational on the Rhine river since 1989. While most of the models mentioned above primarily consider pollutants originating from industrial and municipal wastes,

some also consider pollution originating from diffuse sources such as forestry and agriculture activities or non-sewered residences.

Among the most widely used models is SWAT, which stands for soil and water assessment tool. It allows the simulation of the fate of nutrients and pesticides migrating to water from diffused sources such as agriculture. SWAT is a watershed-scale model developed by Arnold and collaborators for the Agricultural Research Service of the United States Department of Agriculture (USDA) to predict the impact of land management practices on water, sediment and agricultural chemical yields (Arnold and others, 1993). The model combines significant elements of both a physical and semi-empirical nature and can be called a process-based model. Hence, it requires specific information about weather, soil properties, topography, vegetation and land management practices occurring in the watershed. The physical processes associated with water movement, sediment movement, crop growth, nutrient cycling and the like are directly modelled by SWAT using this input data. SWAT is a continuous time model and is not designed to simulate in detail single-events such as floods with hourly time steps.

The objective of SWAT is to predict the effect of management decisions on water, sediment, nutrient and pesticide yields with reasonable accuracy on large, ungauged river basins. The model contains the following components: weather, surface runoff, return flow, percolation, evapotranspiration, transmission losses, pond and reservoir storage, crop growth and irrigation, groundwater flow, reach routing, nutrient and pesticide loading, and water transfer. Interfaces for the model have been developed in Windows (Visual Basic), GRASS and ArcView. SWAT has also undergone extensive validation. For further information, see http://www.brc.tamus.edu/swat.

A number of follow-up models are based on SWAT. For instance, SWIM, which stands for soil and water integrated model, was developed by Krysanova and others (1998, 2000) specifically for climate and land-use change impact assessment in mesoscale and large river basins and at the regional scale. It includes a three-level disaggregation scheme down to hydrotopes and several modified routines, for example, river routing and forest modules, and new routines for impact studies such as a crop generator, climate data interpolation, adjustment of photosynthesis and transpiration to higher CO_2, nutrient retention and a carbon cycle module.

There are numerous models which replicate chemical movement in aquifer systems. Some are bespoke for a particular situation and others are linked to flow models, such as the MT3D link to MODFLOW.

In the case of groundwater, modelling water quality is dependant on understanding the flow regime of the aquifer. Thus, unless the groundwater flow rates and direction and their variability are known, there is little point in attempting to model complex chemical changes in the aquifer. However, by understanding of the chemical processes at work within the aquifer and the distribution of chemical constituents, both natural and anthropogenic, significant insights can be gained on the flow process within the aquifer. Thus the two processes can be used in conjunction to aid overall calibration.

6.3.6.3 Modelling ice formation

The formation of ice in a river begins when the surface layer of water cools down to 0°C. Below the surface of the stream, the water temperature at that time generally remains above 0°C. Thus, forecasting the date of appearance of ice consists of computing the heat exchange at the surface of the water so that the surface layer of the water will cool to 0°C.

Forecasting water temperature should be performed by the stepwise solution of the heat-budget equation, while taking the variables affecting the heat loss into consideration. The heat loss from the water surface is a function of air temperature, wind speed and turbulence of the water. In its most general form, the equation of the heat balance at the air-water interface for a certain interval of time is as follows:

$$\alpha\,(\overline{\theta}_w - \theta_{sw}) + Q = 0 \tag{6.56}$$

where $\overline{\theta}_w$ is the mean temperature of the water mass of the stream θ_{sw}, is the water-surface temperature (in °K), α is the coefficient of heat transfer (Watt/m^2°K) from the water mass to the air-water interface and Q is the heat loss from the water surface in Watt/m^2.

The basis of modern short-term forecasts of the date of initial occurrence of ice on rivers is the method developed by Hydrometeoizdat (1989). This method is based on the inequality between the two heat fluxes:

$$\alpha_n T_{wn} \leq -Q_m^* \quad or \quad T_{wn} \leq -\frac{Q_m^*}{\alpha_n} \tag{6.57}$$

where T_w is the mean temperature of water flow, α_n is the heat-yield coefficient of the water body, Q_m* is the heat loss through the air-water interface and n refers to the time when this inequality appears. The calculation of α_n, T_w, and Q_m* requires knowledge of several meteorological and hydrological variables. The method can be used if air temperature forecasts are available several days ahead. Its accuracy is affected mostly by errors in the anticipated air temperature.

The necessary condition for the beginning of freeze-up is the accumulation of sufficient amounts of floating ice with intensive heat loss so that the merging of ice floes resists the force exerted by the flowing water. This condition is expressed by the following empirical formula:

$$(Q_a)_c = -6.5\,v^2 \left(\frac{b}{\sum Q_a}\right)^{0.8} \qquad (6.58)$$

where $(Q_a)_c$ is the critical, or highest possible, mean daily air temperature on the day of freezing, v is the mean velocity of flow in the reach, b is the river width, and ΣQ_a is the sum of mean daily temperatures from the first day of ice appearance (Buzin and others, 1989). Calculations are made with forecast mean daily temperatures for each day successively until the mean daily air temperature falls below the critical point $(Q_a)_c$, as calculated in equation 6.55. When the critical point is reached, the formation of a frozen section is forecasted.

In operational practice, the full version of the model of ice cover formation, including some form of simplified updating with respect to the specific location and hydrometeorological data, is generally not used. As a rule, model development and applications are tailored to meet user requirements. Thus, the operation of water-management schemes under winter conditions should be based on appropriate reports and forecasts. An ice-oriented hydrological network, which can operate according to forecasting requirements, needs to be implemented according to those principles.

Regular feedback from the water managers to the forecasting centre is also necessary. For the hydropower production it is important to have forecasts of the beginning of intensive frazil ice and slush production. Empirical formulae based on a simplified variant of the theoretical method are generally developed for this purpose. In general, the empirical ratios are represented as nomograms, an example of which is provided in Figure II.6.16.

Short-term forecasting of ice phenomena is based on knowledge of the physical or statistical relationships that exist as necessary conditions for the formation of ice (Hydrometeoizdat, 1989). The physical interpretation of these relationships is based on theories concerning the processes that govern the cooling of a mass of water in natural lakes. These equations are used to determine a critical or threshold air temperature, or the sum of negative air temperatures, which, when exceeded, results in the occurrence of ice cover on a water body. As terms of freezing of a reservoir depend on the heat content of the mass of water, the critical air temperature is determined by using an empirical ratio connecting this air temperature with water supply parameters derived from the water level or streamflow.

6.3.6.4 Modelling ice thickness

In addition to forecasts of the date of ice occurrence and the formation of ice cover, other types of forecasts of the autumn ice phenomena are also issued. Ice thickness forecasts are based on calculations of heat loss. Increases in ice thickness mainly occur on its underside and are determined by the energy state in the water column. Sometimes the ice cover thickness grows from its top surface because of the freezing of water from the melting of the snowpack on the ice surface. Melt water is often accompanied with rainfall. This can also result in an additional amount of water occurring on the ice cover caused by the increased pressure on the ice. Forecasting ice thickness is based on estimates of the difference

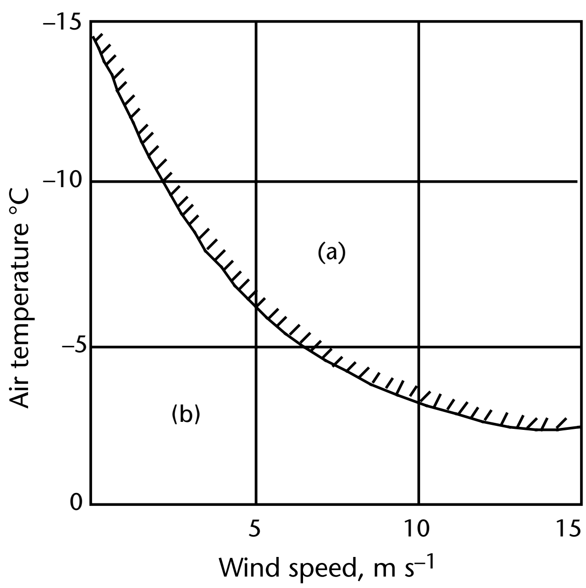


Figure II.6.16. Prediction of frazzle-ice formation: (a) slush possible, (b) no slush

between the inflow of heat from the water column to the bottom surface of the ice and the outflow of heat through the ice surface to the atmosphere. Loss of heat will lead to an increase in thickness of ice of depth:

$$\Delta h_{ice} = \frac{\sum_{1}^{t} B_i \sum_{1}^{t} C_i}{L_{ice}\,\rho_{ice}} \quad (6.59)$$

where Δh_{ice} is the growth in ice thickness in centimetres, $\sum_1^t B_i$ is the flow of heat to the atmosphere from the top surface of a snow-ice cover, $\sum_1^t C_i$ is the inflow of heat to the bottom surface of ice, L_{ice} is the specific heat of ice formation; and ρ_{ice} is the density of ice.

Formulae for calculating and forecasting ice thickness under various conditions of ice cover formation are present in the literature.

6.3.6.5 Modelling ice break-up

One method for forecasting the date of ice break-up is based on estimating the critical sum of degree-days of positive air temperature required for break-up on the river reach in question. To determine this sum, the relationship between break-up and the negative degree-day sum for the winter period is used. To forecast the date of break-up of ice by this method, it is necessary to have an air temperature forecast for a few days in advance. The date of break-up is obtained by calculating the sum of degree-days and comparing it with the critical value, using expected air temperatures for a few days in advance.

Forecasts of the reduction of ice thickness and tensile strength of the ice cover and forecasts of ice break-up for rivers and clarifications from ice of reservoirs are made with ice cover destruction models, such as the models that can be found in Hydrometeoizdat (1989) or through the Bulatov model. The latter is a method of forecasting ice break-up dates on rivers that was developed using a generalized equation, allowing the issuance of medium-term forecasts with a lead-time of ten days. It makes it possible to develop forecasts of ice break-up anywhere, including for rivers with only sparse data (Borsch and others, 1987).

6.3.6.5.1 *Ice break-up on reservoirs*

Break-up of an ice cover on a reservoir results from melt and a gradual decrease in compactness. Under the action of wind, the ice may break into separate ice flows of various sizes, which then start to move as a general drift of ice. The condition for the commencement of a drift of ice is expressed by an inequality of the following form:

$$\varphi d_g^{1/2} \leq CU^2 \quad (6.60)$$

where φ is the compactness of the melting ice (relative bending stress), d_g is the thickness of the ice in centimetres, U is the maximum wind speed over a 24-hour period in m s^{-1} and C is an empirical coefficient that depends on wind speed and is a constant for a given reservoir. For a number of reservoirs in the Commonwealth of Independent States, the value of C was found to be 0.018. The compactness of the ice φ and the thickness d_g when the ice starts to drift are calculated from meteorological elements using heat balance equations. Specific information on applying this method was provided in Hydrometeoizdat (1989).

6.3.6.5.2 *Ice break-up on rivers*

Forecasting the break-up of ice on rivers can be based on models in which the condition for the break-up of the ice cover is determined from the thickness and compactness of the ice and the tractive force of the current. When the forces of resistance become equal to or less than the tractive force, the ice cover breaks up and an ice run begins.

The condition for break-up is expressed by the following relationship:

$$\varphi d_g \leq f(H, \Delta H) \quad (6.61)$$

where φd_g, the product of relative stress of the melting ice and its thickness, is a measure of the compactness of the ice cover at the time of break-up, and H and ΔH are parameters representing the tractive force of the current. H is the height of the water level at the time of break-up and reflects discharge and speed of flow, and ΔH is the rise, up to the time of break-up, in the water level above the minimum winter level H_3, numerically equal to $\Delta H = H - H_3$. As H and ΔH are interrelated in most cases, it is sufficient to consider just one of these quantities in the relationship described in equation 6.61. The quantities are based on forecast and actual data for a few days before break-up. An approximation of the relationship may be expressed as follows:

$$\varphi d_g \leq a + b\,(\Delta H)^2 \quad (6.62)$$

where a and b are empirical coefficients.

For the forecast of ice break-up dates on ungauged rivers, or where there are only short periods of observations available, a forecasting methodology has been developed based on a generalized equation:

$$(\varphi d_g)_{b-i}/(\varphi d_g)_N \leq [1-e^{-(i+1)(Q_{b-i})/(Q_b)_N}]$$
$$(Q_{b-i})/(Q_b)_N + 0.005i + 0.25 \quad (6.63)$$

where $(\varphi d_g)N$ is the average relative durability of the ice on the day of ice break-up, $(\varphi d_g)_{b-i}$ is the relative durability of ice for i days before the ice break-up, Q_{b-i} is the water discharge for i days before the ice break-up, $(Q_b)_N$ is the average discharge on the day of the ice break-up. Calculation and forecast of $(\varphi d_g)N$, $(Q_b)_N$, and d_g are made using specially developed maps, nomograms and tables (Borsch and Silantjeva, 1987).

The model of ice cover break-up allows the development of some additional special forecasts, such as the forecast of maximum permissible loading for ice and forecasts tailored for the deployment of icebreakers.

6.4 MODELLING CHALLENGES

6.4.1 Accuracy and availability of input data

A modelling challenge, related to ungauged basins, is the need to improve the availability and accuracy of the data used in models. This may include the input time series of data such as rainfall and evaporation, and the time-series data used to calibrate or validate model results such as streamflow, groundwater levels and water quality data, as well as the information that is used to estimate model parameter values. If hydrological models are to realize their true potential as operational water resources management tools, it is essential that the information required to apply them successfully be available. The use of processed satellite imagery within modelling research projects has been reported for a number of years and there are examples of such technology being used for operational purposes. However, there is a tremendous potential for the more widespread use of these techniques by water resources management agencies, especially in the developing world where ground-based observations are not being sustained.

Global, or near-global, datasets of a wide range of terrestrial information derived from satellite imagery are becoming increasingly available and accessible. The information available includes relatively static characteristics such as topography, land cover (d'Herbès and Valentin, 1997), as well as time-series variations of parameters such as temperature (Xiang and Smith, 1997), evapotranspiration (Kite and Droogers, 2000), soil moisture (Valentijn and others, 2001) and precipitation (WMO-WCRP, 1986). Many of these have the potential to fill some of the information gaps and provide input data for water resources estimation models. However, several practical considerations need to be addressed if such products are to be used successfully and with confidence:

(a) Hydrological models, calibrated against historical gauged data, may already be in use;
(b) Satellite data have relatively short periods of record;
(c) Ideally, gauged and satellite data need to be used together; therefore, the relationships between the two data sources need to be quantified and clearly understood;
(d) Data should be accessible to water resources practitioners in the developing countries;
(e) The techniques required to make effective use of the data should not be excessively complex or difficult to understand, as the resources available for data analysis and processing are frequently limited in developing countries.

One of the future challenges in hydrological modelling will be to expand the operational use of these techniques. The implication of this is the need to ensure that the results are as accurate and representative as possible. At the very least, it is essential to understand the limitations and error bounds of the model results so that water resources development decisions can be made on the basis of adequate information. The following subsection refers to one of the objectives of the International Association of Hydrological Sciences's initiative for the Decade on Prediction in Ungauged Basins: a reduction in predictive uncertainty. For operational uses of models, it is not only important to achieve a reduction in uncertainty, but also to be able to quantify that uncertainty by having a thorough understanding of the accuracy of the input data.

6.4.2 Ungauged basins

Drainage basins in many parts of the world are either ungauged or inadequately gauged and the situation is worsening because the existing observation networks are in decline. At the same time, water resources are under growing threat in a world that is becoming more populated and where the

demand for water per capita is constantly increasing. Therefore, as the supply of data declines, the need for such data increases. This poses a considerable challenge to the hydrological and water resources communities: that of finding the means to assess and manage water resources with an inadequate supply of data.

Recognition of the need for techniques applicable to ungauged basins is not new. The nineteenth-century rational formula, based on the concept of a runoff coefficient, can be regarded as a precursor of regionalization. Extrapolation from gauged to ungauged sites to solve hydrological problems has been a standard technique in practice. This chapter contains several examples. The synthetic unit hydrographs and geomorphoclimatic unit hydrographs mentioned in 6.3.2.2.5 allow a modeller to estimate a runoff hydrograph for areas with few, if any rainfall and runoff data. An estimation problem is presented in 6.2.3, where an introducion to geostatistics is provided to estimate a value of the variable in an ungauged location, based on a number of values of this variable measured in other locations.

An example of regionalization conducted on a national scale is set out for the United Kingdom in the *Flood Estimation Handbook*, published in 1999, which superseded the *Flood Studies Report* and its supplements. The Handbook explains regionalization of model parameters and extrapolation from gauged to ungauged catchments. It is recommended to conduct flood frequency estimation by statistical analysis of peak flow, annual maxima or peak-over-threshold, or by using a rainfall-runoff approach, if sufficiently long data records are available. While flood data at the subject site are of greatest value, data may be transferred from a nearby site, a donor catchment, a similar catchment or an analogous catchment if there is no donor catchment nearby. Estimation of the index flood – the median annual flood – in the absence of flood peak data can be determined from catchment descriptors. Pooled analysis may be needed for growth curve estimation, dependent on the length of gauged record or the target period such as a 100-year or 10-year flood. The last choice is to estimate parameters for a rainfall-runoff model using only catchment descriptors.

The International Association of Hydrological Sciences, which launched the Decade on Prediction in Ungauged Basins, 2003–2012, aims to achieve major advances in the capacity to make predictions in ungauged basins (Sivapalan and others, 2003). It is hoped that the Decade will bring a reduction in predictive uncertainty and contribute to the development of new theories based on scaling and multi-scaling, complex systems approach, non-linear dynamics and eco-hydrological relationships. This cannot be done without extending the range and scale of observations used in estimating hydrological variables. The initiative is of considerable interest to operational hydrology, and it is hoped that, by the end of the decade, the toolkit of operational techniques for dealing with ungauged basins will have grown considerably.

6.4.3 Coupling of models

With an increasing emphasis on integrated water resources management, it is often necessary to make use of several models to solve practical water resources problems. Examples might include the combined use of water quantity and quality models with systems models and economic impact models. A further example is the use of climate models to generate meteorological inputs to models of basin hydrology. In the past this has been achieved by modelling the different processes separately in series and using the outputs of one model as inputs to the next. This approach has the potential of ignoring many of the feedbacks that exist in complex natural systems. A better approach is to run the models in parallel, whereby the links between processes are coupled at each time step of the simulation and feedback mechanisms are included. Using traditional methods, this involves combining all the algorithms of the separate models into a single model, a substantial development task that precludes the flexibility of selecting different modelling approaches for specific applications. The coupling of models can be facilitated by the development of modelling frameworks that integrate the management of data, geograpical information system visualization tools and model links into a single software package that includes several models. There are a number of such systems available worldwide, all of which have been developed for different purposes. Examples can be found at http://www.epa.gov/waterscience/basins/bsnsdocs.html and Hughes (2004*b*).

A recent innovation, the Open Modelling Interface (OpenMI – see http://www.harmonit.org) represents an attempt to allow models simulating different water-related processes to be linked on a temporal and spatial basis and thus permit the simulation of process interactions. The objective is to simplify the linking of models running in parallel, and operating at different

temporal and spatial scales, through the direct transfer of data between the models. Many existing models are expected to become OpenMI compliant in the near future.

References and further reading

Andersen, T., 1991: AVHRR data for snow mapping in Norway, *Proceedings of the 5th AVHRR Data Users Meeting*, Tromsoe, Norway.

Anderson, E.A., 1973: National Weather Service River Forecast System: Snow Accumulation and Ablation Model, Programs and Test Data. *NOAA NWSHYDRO Technical Memorandum 17*.

Anderson, M. P., D.S. Ward, E.G Lappala and T.A. Prickett, 1992: *Computer models for subsurface water*. Chapter 22 in: Maidment, D. R. (ed.) Handbook of Hydrology, McGraw-Hill, 22–1 to 22–34.

Arnold, J.G., P.M. Allen and G. Bernhardt, 1993: A comprehensive surface-groundwater flow model. *Journal of Hydrology*. 142:47–69.

Baumgartner, M. F., 1988: Snowmelt runoff simulation based on snow cover mapping using digital Landsat-MSS and NOAA/AVHRR data, USDA-ARS, *Hydrology Lab. Tech. Rep*.

Bear, J., 1980: *Hydraulics of Groundwater*. New York, NY: McGraw-Hill College, 1980. ISBN: 0070041709.

Bear, Jacob. 1988: *Dynamics of Fluids in Porous Media*. New York, NY: Dover Publications, 1988. ISBN: 0486656756.

Beldring, S., K. Engeland, L.A. Roald, N.R. Saelthun and A. Voks, 2003: Estimation of parameters in a distributed precipitation – runoff model for Norway. *Hydrology and Earth System Sciences*, 7(3):304–316.

Bergstörm, S., 1992: *The HBV model – its structure and applications*. SMHI Reports RH, No. 4, Norrkping, Sweden.

———, 1995: The HBV model. In Singh, V.P. (ed): Computer Models of Watershed Hydrology, *Water Resources Publications*. Colorado, United States, 443, 476.

Bergstörm, S., J. Harlin and G. Lindstrm, 1992: Spillway design floods in Sweden. I: New guidelines. *Hydrological Sciences Journal*, 37(5):505–519.

Bergström, S., B. Carlsson, M. Gardelin, G. Lindstrm, A. Pettersson and M. Rummukainen, 2001: Climate change impacts on runoff in Sweden – assessments by global climate models, dynamical downscaling and hydrological modelling. *Climate Research*, 16(2):101–112.

Beven, K. J., 1996: A discussion of distributed hydrological modelling. In: M.B. Abbott and J. Ch. Refsgaard, (eds) Distributed Hydrological Modelling, *Water Science and Technology Library*, Vol. 22, Kluwer, Dordrecht, 255–278.

Biswas, A.K. (ed.), 1981: *Models for Water Quality Management*. McGraw-Hill, New York.

Borsch S.V. and T.P. Silantjeva, 1987: Method of the short-term and medium-term forecast of opening of the rivers on the basis of the generalized dependence. The methodical instructions. Moscow, Hydrometeorological Center of the Union of Soviet Socialist Republics. pp. 28.

Box, G.E.P. and G.M. Jenkins, 1970: *Time Series Analysis, Forecasting and Control*, San Francisco, Holden-Day.

Brun, E., P. David, M. Sudak and G. Brunot, 1992, A numerical Model to Simulate snow-cover stratigraphy for operational avalanche forecasting, *Journal of Glaciology*, 38(128), 13–22.

Carroll, S.S., G.N. Day and T.R. Carroll, 1993: Incorporating airborne data into spatial model used to estimate snow water equivalent, Geographic Information Systems and Water Resources, *Journal of the American Water Resources Association*, March, pp. 259–264.

Cigizoglu, H. K., 2003: Estimation, forecasting and extrapolation of river flows by artificial neural networks. *Hydrololgoical Science Journal*, 48(3):349–361.

d'Herbès, J.M. and C. Valentin, 1997: Land surface conditions of the Niamey region: ecological and hydrological implications. *Journal of Hydrology*, 188–189, 18–42.

DeWeist, J.M., 1965: *Geohydrology*. John Wiley & Sons, 366 pp.

DHI (Danish Hydraulic Institute), 1985: *Introduction to the SHE-European Hydrologic System*, Horsholm.

Domenico, Patrick A. and W. Scgwartz Franklin, 1998: *Physical and Chemical Hydrogeology*. New York, NY: John Wiley & Sons Inc., 1998. ISBN: 0471597627.

Donald J.R., E.D. Soulis, N. Kouwen, A. Pietroniro, 1995: A land cover-based snow cover representation for distributed hydrologic models. Water Resources Research. 31, No. 4: pp. 995–1009.

Dooge, J.C.I., 1973: Linear theory of hydrologic systems. *Technical Bulletin No. 1468*, Agricultural Research Service, United States Department of Agriculture, Washington, DC.

Eagleson, P.S., 1970: *Dynamic Hydrology*, New York McGraw-Hill.

Engman, E.T. and R.J. Gurney, 1991: *Remote-sensing in hydrology*, London, Chapman and Hill, 225 pp.

Environment Agency, 2002: *Groundwater Resources Modelling*, Guidance Notes and Template Project Brief, R & D Guidance Notes W213, June 2002.

Fetter, C. W., 1998: *Contaminant Hydrogeology*. 2nd ed. Upper Saddle, NJ, Prentice Hall, ISBN: 0137512155.

Freeze, Alan R. and A. Cherry John, 1979: *Groundwater*. Englewood Cliffs, NJ: Prentice Hall. ISBN: 0133653129.

Hedstrom N.R. and J.W. Pomeroy, 1998: Measurements and modelling of snow interception in the boreal forest. *Hydrological Processes* 12(10–11):1611–1625.4.

Hillel, Daniel, 1980: *Fundamentals of Soil Physics*. New York, NY, Academic Press, 1980. ISBN: 0123485606.

Hipel, K.W., A.L. McLeod and W.C. Lennox, 1977: Advances in Box-Jenkins modelling. Part I: Model construction. *Water Resources Research*, 13:567–575.

Hirsch, R.M. and J.R. Slack, 1984: A nonparametric trend test for seasonal data with serial dependence: *Water Resources Research*, pp. 727–782.

Hughes, D.A., 2004*a*: Incorporating groundwater recharge and discharge functions into an existing monthly rainfall-runoff model. *Hydrololgoical Science Journal*, 49(2):297–311.

Hughes, D.A. (ed.), 2004*b*: SPATSIM, an integrating framework for ecological reserve determination and implementation. Water Research Commission Report No. TT 245/04, United States.

Hughes, D.A. and W. Metzler, 1998: Assessment of three monthly rainfall-runoff models for estimating the water resource yield of semiarid catchments in Namibia, *Hydrololgoical Science Journal*, 43(2):283–297.

Hurst, H.E., 1951: Long-term storage capacity of reservoirs. Transactions, *American Society of Civil Engineers*, 16:770.

Hydrometeoizdat, 1989: *Guide on the hydrological forecasts*. The ice forecasts. Vol. 3. Leningrad, Hydrometeoizdat, pp. 168 (in Russia).

Institute of Hydrology, 1999: *Flood Estimation Handbook (FEH)*: (in five volumes), Wallingford, Oxfordshire, United Kingdom.

Kalinin, G. P. and P.I. Miljukov, 1958: *Approximate methods for computing unsteady movement of water masses*. Transactions, Central Forecasting Institute, Issue 66 (in Russian)

Keller, M., 1987: Ausaperungskartierung mit Landsat-MSS Daten zur Erfassung oekologische Einflussgnessen in Gebirge, Remote-sensing Series, No. 10, PhD Thesis, University of Zurich.

Kitanidis, P.K., 1992: Analysis of Macrodispersion through Volume–Averaging: Moment Equations, *Stochastic Hydrology and Hydraulics*, 6:5–25.

Kite, G.W. and P. Droogers, 2000: Comparing evapotranspiration estimates from satellites, hydrological models and field data. *Journal of Hydrology*, 229(1–2):3–18.

Krysanova, V., Müller-Wohlfeil D.I., and Becker A., 1998: Development and test of a spatially distributed hydrological/water quality model for mesoscale watersheds. *Ecological Modelling* 106:261–289.

Krysanova, V., E. Wechsung, J. Arnold, R. Srinivasan and J. Williams, 2000: SWIM (Soil and Water Integrated Model). User Manual, PIK Report 69. Potsdam Institute for Climate Impact Research.

Kundzewicz, Z. W. and A. Robson, 2004: Change detection in hydrological records – a review of the methodology. *Hydrololgoical Science Journal*, 49(1):7–19.

Leaf, C.F., and G.E. Brink, 1973: *Hydrologic simulation model of Colorado subalpine forest*, US Forest Service, Res.Pap. RM-107, Fort Collins, CO.

Leavesley, G.H. and L.G. Stannard, 1990: Application of remotely sensed dataon a distributed parameter-watershed model, *Proceedings of the Workshop on Applications of Remote-sensing in Hydrology*, Editors: Kite, G.W. and Wankiewicz, A., National Hydrology Research Institute, Saskatoon, Saskatchewan, Canada.

Lindström, G., 1997: A simple automatic calibration routine for the HBV model. *Nordic Hydrology*, 28(3):153–168.

Lindström, G. and J. Harlin, 1992: Spillway design floods in Sweden. II: Applications and sensitivity analysis. *Hydrololgoical Science Journal*, 37(5):521–539.

Lindström, G. and S. Bergström, 1992: Improving the HBV and PULSE-models by use of temperature anomalies. *Vanneti Norden*, 25(1):16–23.

Lindström, G., Johansson, B., Persson, M., Gardelin, M. and Bergström, S., 1997: Development and test of the distributed HBV-96 model. *Journal of Hydrology*. 201:272–288.

Maidment, D. R. 1992: *Handbook of Hydrology*. New York, McGraw-Hill.

Mandelbrot, B.B. and J.R. Wallis, 1968: Noah, Joseph, and operational hydrology. *Water Resources Research*, 4(5):909–918. Available at the address http://www.math.yale.edu/mandelbrot/web_pdfs/051noahJoseph.pdf

Marks, D., 1988: Climate, energy exchange and snowmelt on Emerald Lake watershed, Sierra Nevada, Unpublished PhD Dissertation, University of California, Santa Barbara, CA, 149 pp.

Marsily, Ghislain De., 1986: *Quantitative Hydrogeology: Groundwater Hydrology for Engineers*. Orlando, FL, Academic Press, 1986. ISBN: 0122089162.

Martinec, J. and A. Rango, 1987: Interpretation and utilization of areal snow cover data from satellites, *Annual Glaciology*, 9:166–169.

McDonald, M.G. and A.W. Harbaugh, 1988: A modular three-dimensional finite-difference groundwater flow model: US Geological Survey, *Techniques of Water Resources Investigations*, Book 6, Chapter Al, 586 pp.

Mejia, J.M., I. Rodriguez-Iturbe and D.R. Dawdy, 1972: Streamflow simulation, (2) The broken line process as a potential model for hydrologic simulation. *Water Resources Research*, 8(4):931–941.

Miller, W.A., M.B. Shasby, W.G. Rhode and G.R. Johnson, 1982: *Developing inplace data bases by incorporating digital terrain data into the Landsat classification process*. Place Resource Inventories:

principles and practices, Proc. Workshop, August 4–14, 1981, University of Maine, Orono, Sponsored by the American Society of Photogrammetry.

Minns, A.W. and M.J. Hall, 1996: Artificial neural networks as rainfall-runoff models, *Hydrololgoical Science Journal*, 41(3):399–417.

Ostrem, G., T. Anderson and H. Odegaard, 1991: Operational use of satellite data for snow inventory and runoff forecast, Satellite Hydrology, *American Water Resources Association*, Minneapolis, MN, pp. 230–234.

Pomeroy J.W. and E. Brun, 2001: Physical Properties of Snow. In Snow Ecology: An Interdisciplinary Examination of Snow-covered Ecosystems. H.G. Jones, J.W. Pomeroy, D.A. Walker and R.W. Hoham (eds). Cambridge University Press: Cambridge; pp. 45–126.

Price, Michael, 1985: *Introducing Groundwater*. George Allen & Unwin: London, United Kingdom, 1985. 195 pp.

Rango, A., 1993: Snow hydrology processes and remote-sensing, *Hydrological Processes*, 7:121–138.

Rango, A., and van Katwijk. 1990: Development and testing of a snowmelt-runoff forecasting technique, *Water Resources Bulletin*, 26:135–144.

Rango, A., V.V. Salomonson and J.L. Foster, 1977: Seasonal streamflow estimation in the Himalayan region employing meteorological satellite snow cover observations, *Water Resources Research*, 13:109–112.

Refsgaard, J.C. and M.B. Abbott, 1996: The role of distributed modelling in water resources management. In: M.B. Abbott and J. Ch. Refsgaard, (eds), 1996: Distributed Hydrological Modelling, *Water Science and Technology Library*, Vol. 22, Kluwer, Dordrecht.

Robson, A.J., T.A. Jones and D.W. Reed, 1998 A study of national trend and variation in United Kingdom floods. *International Journal of Climatology*, 18:165–182.

Rodriguez-Iturbe, I. and B. Valdes, 1979: The geomorphologic structure of hydrologic response, *Water Resources Research*, 15(6):1409–1420.

Rodriguez-Iturbe, I., J.M. Mejia and D.R. Dawdy, 1972: Streamflow simulation, (1) A new look at Markovian models, fractional Gaussian noise, and crossing theory. *Water Resources Research*, 8(4):921–930.

Sahimi, Muhammad, 1995: *Flow and Transport in Porous Media and Fractured Rock*. Weinheim, Germany: Wiley-VCH Verlag GmbH, 1995. ISBN: 3527292608.

Salas, J.D., 1992 Analysis and modelling of hydrological time series. Chapter 19 in: Maidment, D. R. (ed.) *Handbook of Hydrology*, Mc Graw-Hill, 19–1 to 19–72.

Scheidegger, Adrian, 1960: *The Physics of Flow through Porous Media*, University of Toronto Press.

Shaw, E.M., 1994: *Hydrology in Practice*, third edition. London, Chapman and Hall.

Singh, V.P., 1988: *Hydrologic Systems*. Rainfall-Runoff Modeling, Vol. I, Englewood Cliffs, Prentice Hall.

Sivapalan, M., K. Takeuchi, S. Franks, D. Schertzer, P.E. O'Connell, V.K. Gupta, J.J. McDonnell, J. W. Pomeroy, S. Uhlenbrook, E. Zehe and V. Lakshmi, 2003: IAHS Science Decade on Prediction in Ungauged Basins (PUB), 2003–2012: Shaping an exciting future for the hydrological sciences. *Hydrololgoical Science Journal*, 48(6):857–880.

SNA, 1995: *Climate, Lakes and Rivers*, The National Atlas of Sweden, Almqvist and Wiksell International, Stockholm.

Storm, B. and A. Refsgaard, 1996: Distributed physically based modelling of the entire land phase of the hydrological cycle. In: M.B. Abbott and J. Ch. Refsgaard, (eds): *Distributed Hydrological Modelling*, Water Science and Technology Library, Vol. 22, Kluwer, Dordrecht.

Sugawara, M., E. Ozaki, L. Watanabe and S. Katsuyama, 1974: *Tank Model and its Application to Bird Creek*, Wollombi Brook, Bikin River, Kitsu River, Sanaga River and Nam Mune. Research Note of the National Research Center for Disaster Prevention, Science and Technology Agency, Tokyo, Japan.

Tayfur, G., 2002: Artificial neural networks for sheet sediment transport. *Hydrololgoical Science Journal*, 47(6):879–892.

Todd, David Keith, 1980: Groundwater Hydrology. New York, NY. John Wiley & Sons Inc. ISBN: 047187616X.

Todini, E. and J.R. Wallis, 1978: A Real-time Rainfall Runoff Model for an On-line Flood Warning System. AGU Chapman conference on applications of kalman filtering theory and techniques to hydrology, hydraulics and water resources, Pittsburgh, Pennsylvania, 22–24 May 1978.

Twedt, T. M., J.C. Schaake and E.L. Peck, 1977: National Weather Service extended streamflow prediction. Proceedings of the western snow conference, Albuquerque, New Mexico, 19–21 April 1977.

US Army Corps of Engineers, 1960: *Runoff from Snowmelt*. Engineer Manual 1110-2-1406, US Department of the Army, Washington, DC.

US Environmental Protection Agency, 1989: *EPA Groundwater Handbook*. Rockville, Maryland, Government Institutes, Inc. 212 pp.

Valentijn R.N.P., R. Hoeben, N.E.C. Verhoest and E.P. De Troch, 2001: The importance of the spatial patterns of remotely sensed soil moisture in the improvement of discharge predictions for small-scale basins through data assimilation. *Journal of Hydrology*, 251(1–2):88–102.

Walton, W., 1970: *Groundwater Resource Evaluation*, McGraw-Hill, 664 pp.

World Meteorological Organization, 1986: *Intercomparison of Models of Snowmelt Runoff*.

Operational Hydrology Report No. 23, WMO-No. 646, Geneva.

———, 1987: *Intercomparison of Conceptual Models Used in Operational Hydrological Forecasting*. Operational Hydrology Report No. 7, WMO-No. 429, Geneva.

———, 1988: *Analysing Long Time Series of Hydrological Data with respect to climate variability and change*, WCAP-3, WMO/TD-No. 224, Geneva.

———, 1990: *Hydrological Models for Water-Resources System Design and Operation*. Operational Hydrology Report No. 34, Geneva.

———, 1991*a*: Simulated Real-time Intercomparison of Hydrological Models. Operational Hydrology Report No. 38, WMO-No. 779, Geneva.

———, 1991*b*: Hydrological Aspects of Accidental Pollution of Water Bodies. Operational Hydrology Report No. 37, WMO-No. 754, Geneva.

———, 1992*a*: *A study of current approaches to modelling of hydrological times series with respect to climatic variability and change* (G. Cavadias). World Climate Programme – Water, Project A2. WCASP–23 WMO/TD-No. 534, Geneva.

———, 1992*b*: *Snow Cover Measurements and Areal Assessment of Precipitation and Soil Moisture*. Operational Hydrology Report No. 35, WMO-No. 749, Geneva.

———, 1994: Applications of remote-sensing by satellite, radar and other methods to hydrology, Operational Hydrology Report No. 39, WMO-No. 804, Geneva.

——— , 2000: *Detecting Trend and Other Changes in Hydrological Data*. World Climate Programme – Water (Z.W. Kundzewicz and A. Robson, eds) World Climate Programme Data and Monitoring, WCDMP-45, WMO/TD-No. 1013, Geneva.

———, 2003: *Integration and Coupling of Hydrological Models with Water Quality Models: Applications in Europe* (B. Arheimer and J. Olsson). WMO Technical Reports in Hydrology and Water Resources, No. 75. WMO/TD-No. 1174. Geneva.

———/WCRP, 1986. *Report of the Workshop on Global Large Scale Precipitation Datasets for the World Climate Research Programme*. WCP-111, WMO/TD-No. 94, WMO, Geneva, 45pp.

Xiang, X. and E.A. Smith, 1997: Feasibility of simultaneous surface temperature-emissivity retrieval using SSM/I measurements from HAPEX-Sahel. *Journal of Hydrology*, 188–189, 330–360.

Yue, S. and P. Pilon, 2004: A comparison of the power of the t test, Mann-Kendall and bootstrap tests for trend detection. *Hydrololgoical Science Journal*, 49(1):21–37.

Yue, S., P. Pilon and B. Phimney, 2003: Canadian streamflow trend detection: impacts of serial and cross-correlation. *Hydrololgoical Science Journal*, 48(1):51–63.

Yue, S., P. Pilon and G. Cavadias, 2002: Power of the Mann-Kendall and Spearman's rho tests for detecting monotonic trends in hydrological series. *Journal of Hydrology*, 259:254–271.

CHAPTER 7

HYDROLOGICAL FORECASTING

7.1 INTRODUCTION TO HYDROLOGICAL FORECASTING [HOMS J]

7.1.1 Scope

A hydrological forecast is the estimation of future states of hydrological phenomena. They are essential for the efficient operation of water infrastructure and the mitigation of natural disasters such as floods and droughts. In addition, they are becoming increasingly important in supporting integrated water resources management and reducing flood-induced losses.

Describing and predicting future water states can be categorized on the basis of how far into the future the event is forecast to occur. For instance, forecasts for various hydrological elements such as discharges, stages and velocities can be made from the start of the forecast up to different times in the future. The Technical Regulations provide the following classification:

(a) Short-term hydrological forecasts, which cover a period of up to two days;
(b) Medium-range hydrological forecasts, which apply to a period ranging from 2 to 10 days;
(c) Long-range hydrological forecasts, which refer to a period exceeding 10 days.

This section discusses the importance and necessity of establishing an end-to-end hydrological forecasting programme, while 7.1.5 provides an introduction to the communications technology used to collect data and distribute critical forecasts and warnings to its users, and 7.2 describes the data requirements for hydrological forecasting. An overview is provided in 7.3 of the various forecasting techniques available, from simple index models to robust hydrological forecasting systems. Forecasting of flash floods (see 7.4) and snowmelt (see 7.6) have been dealt with in greater detail because there is a need for guidance material on those issues. Finally, water supply forecasts are covered briefly in 7.5. The discussion of hydrological forecasts in this chapter will be limited to predicting quantities of water.

7.1.2 Hydrological forecast operations

A hydrological forecasting service is composed of trained hydrological forecasters working with a combination of real-time and historical data inputs, which can include use of radar and satellite as well as in situ data, communications hardware and software, hydrological models or modelling systems, meteorological models or model product or inputs and computer hardware. There are many ways to configure a hydrological forecasting service. There are, however, a critical number of factors that are necessary to ensure reliable delivery of a service meeting the needs of a diverse user community.

The operations concept of a hydrological forecasting service defines how the operational forecast service will operate on a day-to-day basis, as well as during fooding conditions. It covers the following points:

(a) The mission and legal mandate of the organization;
(b) The users and the required products or services;
(c) Deadlines for dissemination;
(d) How the hydrological forecasting service is organized;
(e) The hydrometeorological data network and how it operates;
(f) How the hydrologist will interact with the meteorological forecasting office;
(g) Communications hardware and software used to receive data and information as well as disseminate forecasts;
(h) How forecast products are produced;
(i) What policies and standard operating procedures will be followed to ensure best practices during routine and emergency conditions;
(j) The outreach of the hydrological forecasting service through the education and training of policymakers, emergency operations staff and the general public.

Sample products should be readily available for potential customers.

The mission and legal mandate of the hydrological forecasting service needs to be clearly defined. It is important that only one official source of forecast and warnings be authorized by law. Multiple sources of forecasts can result in conflicting information that produces confusion and reduces the possibility of effective response.

The principal users of warning products are national, regional and local emergency management or civil defence organizations, the media, agriculture, industry, hydropower organizations, flood control managers, water transportation and municipal water supply organizations and the public. The requirements of hydrological data, forecast products and warnings vary according to the targeted user community. It is essential for the hydrologist to understand user requirements so that data and forecast products can be tailored to meet their needs. There are many segments of a national economy, such as transportation, emergency management, agriculture, energy and water supply, that have unique needs for such information. Recognizing these needs and providing data, forecasts and products to meet them ensures that the hydrological forecasting service is of greatest benefit to the community. Sophisticated users, such as hydropower organizations, require hydrometeorological data, forecasts, inflow hydrographs and analyses to support the generation of electricity, while emergency management operations require simpler but more urgent forecasts and warnings.

The network, including stream gauges, precipitation gauges and the associated meteorological network, should be defined, taking into account the availability of data from all sources, such as the radar network and satellite downlink products. However, the continuous availability of such products must be established before they are used on a regular basis in national hydrological forecasting services. Close cooperation between meteorological forecasting services and hydrological forecasting services is essential. The procedure, or system definition, for the acquisition of data and forecasts, as well as analysis, are needed as input to hydrological forecasts and should be defined in the operations concept. Communications hardware and software used in flood forecasting systems depend on the infrastructure available in the country concerned. However, modern data communication systems, such as satellite and the Internet, provide a variety of choices and should be utilized appropriately.

It is important to assess staff requirements such as the number of technicians or professionals needed to run the centre during routine and emergency operations. Their roles and responsibilities, working hours and the continuous training needs of forecasters should also be addressed.

Hydrological forecasting programmes must be reliable and designed to operate during the most severe floods. The greatest benefits for an effective hydrological forecasting programme occur when flooding is severe, widespread and/or sudden. Normally, there is a greater strain on resources during extreme events such as floods. The operation of the centre during extreme events must be well defined. In such instances, there is generally an increase in data flow and staffing needs, as more products must be delivered to more users with short deadlines. Frequently the hours of operation must be expanded to meet higher demands for service.

During routine conditions, the staff of a hydrological forecasting service collect data and quality-control information, receive and analyse meteorological forecasts, run hydrological models and forecasting systems, assess present and future hydrological conditions and produce forecast products for distribution to users. During non-forecasting portions of the day, hydrologists update data such as rating curves, evaluate operational performance, re-calibrate models and seek further means of improving the accuracy and timeliness of future forecasts.

It is never possible to achieve continuous,100 per cent reliability of hardware, software and/or power for operations even with dependable maintenance programmes. Therefore, a hydrological forecasting service must establish backup procedures to safeguard future operations of all components: data collection; forecasting system operations, including backup of hardware, software and data; forecast dissemination and other communications systems; power, uninterruptible power supply and backup generators; and provision of an alternate site for operations if the location of forecast centre itself is damaged.

The key to making a forecast centre operationally reliable is to establish a robust maintenance programme. Unfortunately this can be an expensive undertaking, especially if the network is spread out and difficult to access. All hardware and software must be routinely maintained, otherwise the system may not function when most needed. In some countries, the hydrological forecasting service includes a system administrator, who is responsible for maintaining the communications and forecasting system.

7.1.3 End-to-end hydrological forecasting systems

Today's hydrological forecasting systems are affordable and powerful. The degree of success in using these systems generally depends on the amount of training received by the hydrologists employing them. These systems are capable of

producing forecasts for floods that occur in a few hours to seasonal probabilistic outlooks many months in advance for larger river basins.

Establishing a viable hydrological forecasting and warning programme for communities at risk requires the combination of meteorological and hydrological data, forecast tools and trained forecasters. Such a programme must provide sufficient lead time for individual communities in the floodplain to respond. In case of flood forecasts, lead time can be critical in reducing damage and loss of life. Forecasts must be sufficiently accurate to promote confidence so that communities and users will take effective action when warned. If forecasts are inaccurate, credibility is reduced and an adequate response is not made.

Experience and lessons from the past have demonstrated that an end-to-end hydrological forecasting and response system (see Figure II.7.1) consists of the following steps, which must be linked to achieve reduction in flood losses:
(a) Data collection and communication;
(b) Hydrological forecasting and forecast product generation;
(c) Dissemination of forecasts to users;
(d) Decision-making and support;
(e) Action taken by users.

The interaction of the technological components of the integrated end-to-end hydrological forecasting system can be represented as a chain composed of many links. Each link must be fully functional to benefit the user community or population at risk. As with links in a chain, should one link not be functioning properly, the entire system breaks down. In other words, if a perfect flood forecast is generated but does not reach the population at risk, or no capabilities to take preventive action exist, then the forecast system does not serve its desired purpose.

7.1.4 Uncertainity and probabilistic forecasts

In general, the primary objective of hydrological forecasting is to provide maximum lead time with sufficient accuracy so that users may take appropriate action to mitigate losses or optimize water management decisions. All forecasts contain uncertainty and one of the most successful ways of dealing with this is the use of ensembles. The uncertainty associated with a hydrological forecast starts with the meteorology. Given that all mesoscale atmospheric models attempt to model an essentially chaotic atmosphere, meteorology has been seen as the primary source of uncertainty for some years. In addition, hydrological model parameters and the model mechanics also contribute to the associated uncertainty or error in forecasts. Adequacy of data is generally the main limiting factor. If only observed hydrological data are used to generate forecasts, lead times may be so short that the utility of forecasts to users is of little value. By coupling hydrological models with meteorological forecasts that are the result of meteorologists implementing global and regional numerical weather prediction models and accounting for local climatological conditions, streamflow forecasts can be extended from many days to weeks in the future. Although coupling of models can indeed extend the lead time for users, it also increases forecast uncertainty.

Climatological or seasonal forecasting has now become a useful tool for managing water and reducing the risk of flooding. Extreme events are correlated with major changes in atmospheric and ocean circulation patterns; once such patterns can be identified, the potential for a lesser or greater degree of storm activity can be forecast. This information can then be used to improve emergency response and increase the degree of readiness of forecasting agencies.

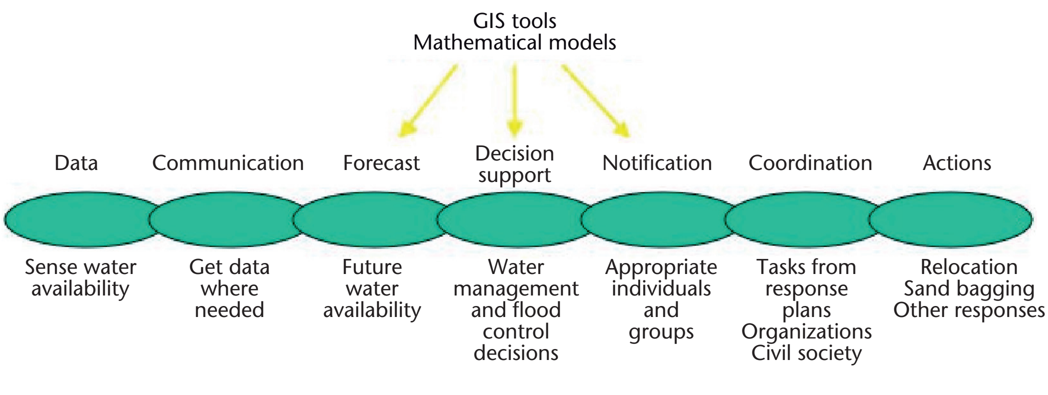


Figure II.7.1. Integrated flood forecasting, warning and response system in integrated water resources management: a critical chain of events and actions

When the probability of an extreme flooding event is forecast to be greater than normal, certain measures can be taken in anticipation of the events, for example, stockpiling sandbags, emergency food and water supplies, and moving high-value stored crops or goods from flood-prone areas. This is also a good time to create awareness among the public as to the potential for flooding, highlighting the actions that the public and others should take, and to carry out emergency-response exercises to test the degree of readiness. In some cases, emergency measures such as the temporary raising of flood protection barriers may be warranted. Recent developments in computing power have allowed global and regional atmospheric models to increase their spatial resolution. Local area non-hydrostatic models, for instance, have been successfully reduced to a spatial scale of approximately one kilometre. In addition, smaller-scale processes, such as convection and orographic enhancement, have been modelled more effectively.

Probability forecasting should not be confused with forecast error. The latter is internal to the model and data, and represents the error caused by model inadequacy and data error. Perhaps the best way to distinguish between them is to view probability forecasting as an expression of the range of outcomes that are possible in light of the conditions that may arise before the forecast date, whereas forecast error is a totally undesirable feature of the shortcomings of the state of forecasting science and of the available data.

The primary mechanism used to incorporate uncertainty directly has been to perturb the initial conditions of the non-linear partial differential equations describing the atmosphere using mesoscale convective system approaches. However, most methods currently in use are suboptimal and still rely on a judicious choice being exercised by the forecaster. The ensemble Kalman filter is widely used to propagate and describe forecast uncertainty. The European Centre for Medium-Range Weather Forecasting (http://www.ecmwf.int) and other international agencies have been investigating the use of mesoscale convective system-based ensembles in recent years and a large-scale intercomparison hydrological ensemble prediction experiment, which was launched in 2005. While this approach is indeed promising, it has yet to be proven, and a considerable amount of work will be required to develop procedures for the propagation of uncertainty through complex model systems.

7.1.5 Dissemination of forecasts and warnings

Forecasts lose value with time. The faster data and forecasts can be sent to users, the more time can be applied to response, thereby saving lives, reducing property damage and enhancing the operation of water resources structures. Dissemination of forecasts and warnings to communities and villages at risk of flooding is frequently a weak link in the end-to-end chain. Significant progress in communications technology allows for the rapid transmission of data, forecasts and information over large distances and to remote locations.

The delivery of hydrological products from a forecast service can be categorized as normal daily forecasts and non-routine urgent forecasts. Many users require the routine transmission of data and forecasts on a daily basis, in the form of a hydrological bulletin. Information is generally given for key rivers, reservoirs and other water bodies of interest to the region. Daily bulletins vary in their composition and frequently include information about current values and trends of stages, discharges, tendency of stages and discharges, water temperature, reservoir data such as pool and discharge, precipitation, hydrological forecasts and ice conditions, if prevalent. Figure II.7.2 provides an example of such a bulletin.

Many routine hydrological products can be produced based on user needs. Water supply forecasts and flow summaries can be issued on a weekly and monthly basis. These summaries frequently provide figures and data for key locations in river basins that may include medium- and long-range forecasts with lead times of weeks, months, or seasons. Distribution of routine hydrological products should be as widespread as possible, since many types of users can benefit from data and forecasts. Opening up data and forecasts to many users enhances the value of forecasting services and builds a constituency for such services, which is necessary if they are to sustain operations in the future.

The Internet is the best means of disseminating information. Although communication is hindered by bandwidth limitations in many developing countries, its use and accessibility is improving. Hydrological forecasting systems can make use of this channel of communication in their dissemination strategies for hydrological products. Other means of distributing products are the public media, use of continuous radio broadcasts and fax.

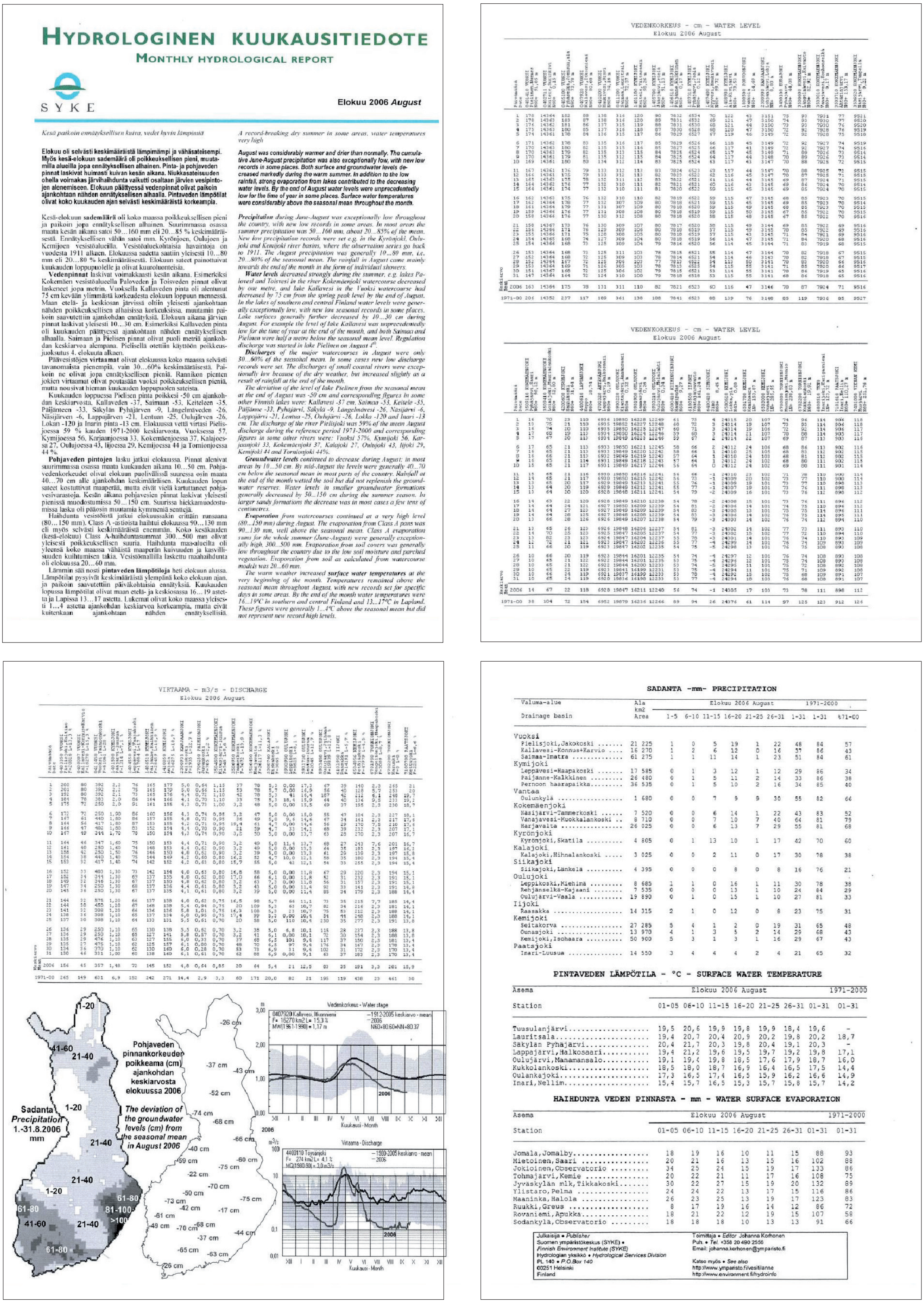


Figure II.7.2. Example of a hydrological bulletin (Finnish Environment Institute)

The composition and distribution of forecast and warning products for medium-term extreme events require that input data be assembled rapidly and that forecasts be made and reach the population at risk in enough time to trigger response measures that will minimize the impacts. Hydrological forecasting centres should explore all available communication channels to reach the specific population at risk. Communication media commonly used are direct transmission lines, satellite, radio and landline to emergency operation centres and radio and television stations.

Under emergency conditions such as flooding, warning products should clearly identify the type of hydrological threat, the location of the predicted event, namely the rivers and streams involved, the magnitude of the event expected, such as the peak flood level at critical locations, the forecast time of occurrence of the peak and, if possible, when the river is expected to fall below warning or danger level. Further details, such as what portion of the infrastructure will be affected by the event, should be provided, if possible. Such information provides emergency response units with locations where action needs to be taken to conduct evacuations and road closures. As more data and information become available, flood-warning products should be updated and disseminated to the media and emergency-response officials.

Advances in coupling hydrological models with expanding geographical information datasets have resulted in the development and implementation of high-visual hydrological forecasting products. This new class of hydrological products shows flood inundation produced by models linked to high-resolution digital elevation model data. By linking that data with hydrological model forecast elevations computed for river channels, the area of flood inundation for the flood plain can be overlaid on top of detailed digital maps of human infrastructure showing how forecast flooding will impact a given location. An illustration of a flood map product is provided in Figure II.7.3.

7.1.6 Decision support

Organizations responsible for water resources management use decision-support tools to provide guidance for the operation of infrastructure. Forecasts for water management are needed to plan effective use of water, ranging from hydroelectric power generation to water supply and irrigation. Measures taken by managers can have significant negative consequences if future water availability is not considered. If hydrological forecasts are available, water resources managers can operate water supply systems to better meet water demand and minimize the potential for conflict.

Flood losses can be reduced if communities and countries invest in flood preparedness and response planning prior to the occurrence of the event. Emergency service organizations are responsible for establishing flood-response plans that outline the role of various national, provincial and local organizations in protecting life and property. This link in the end-to-end chain includes setting up evacuation routes, educating the population at risk of the

Figure II.7.3. Flood map inundation forecast for Hurricane *Mitch* flood in Tegucigalpa, Honduras

flood hazards and establishing procedures and training personnel to respond well in advance of the occurrence of a flood.

The perfect flood forecast has no value unless steps are taken to reduce losses. In the end-to-end process, data and forecasts must be produced as quickly as possible to give users time to take action. In the case of flooding, especially flash flooding, time is critical. Understanding users' needs and how forecasts are used is important. There are a wide variety of users, ranging from federal response agencies to local governments, that have different roles and needs in responding to and mitigating losses. Users must understand the forecast or warning message for appropriate response to occur.

7.1.7 Cooperation with the National Meteorological Service

Although a few countries have a combined meteorological and hydrological service, in most cases the meteorological and water management authorities are separate. Indeed, they are seldom within the same government department or ministry. The provision of good weather forecast information, particularly in relation to severe precipitation events, is a vital part of a flood forecasting and warning operation. It is therefore important that close cooperation be developed between the National Meteorological Service and the flood forecasting service.

Generalized meteorological forecasts are of little use to hydrologists; therefore, an initial step in developing cooperation should be taken to decide where value can be added to the meteorological information and how it can be structured to meet hydrological requirements. This will be largely a matter of improving the information on rainfall forecasts in terms of quantity – quantitative precipitation forecasts – timing and geographic distribution. Typical forecast products are as follows:

(a) Routine forecasts made on a daily basis, giving information on rainfall, temperature and weather for 24–48 hours, and an outlook period of some 3–5 days;
(b) Event forecasts, particularly forecasts and warnings of severe events, such as heavy rainfall, snow and gales, which have hydrological impacts. These should provide good quantitative and areal information over a lead time of 12–36 hours;
(c) Outlook forecasts for periods of weeks or months, or particular seasons. These are useful for planning purposes, especially with regard to drought or cessation of drought conditions. Some national meteorological services, such as those of South Africa, Australia and Papua New Guinea, provide forecasts of El Niño activity where this is known to have direct impacts on weather patterns.

The national meteorological service and the hydrological agency must agree on the structure and content of forecast and warning products. This is usually achieved by an evolutionary or iterative process over time.

It is also useful for other weather service products to be provided to the hydrological agency. The most common products are satellite imagery and rainfall radar information. The information is transferred by dedicated data feeds, which will have a higher degree of reliability than information transferred through public service networks or available from Websites. A direct arrangement for data transfer by the national meteorological service also should ensure that updates are automatically provided, for example, every 3–6 hours for satellite imagery and every 5–10 minutes for radar scans. By using satellite and radar information, hydrological agency staff can make their own assessment and judgement of the current and immediate future weather situation. It is important that staff be adequately trained to do this; the national meteorological service has an important role in providing the necessary training through introductory and updating courses.

The aforementioned arrangements and facilities should be covered by a formal service agreement defining levels of service to be achieved in timeliness of delivery and accuracy of forecasts. The service agreement should also include the costs of service provision. In this manner, both parties can define the economic costs and benefits in providing the service, and the value that the service may have in impact management.

7.2 DATA REQUIREMENTS FOR HYDROLOGICAL FORECASTS

7.2.1 General

Data requirements for hydrological forecasting depend on many factors:
(a) Purpose and type of forecast;
(d) Basin characteristics;
(b) Forecasting model;
(c) Desired degree of accuracy of forecast;

(e) Economic constraints of the forecasting system.

Data requirements vary considerably according to the purpose of the forecast. Operating a reservoir requires reservoir inflow forecasts relating to short intervals of time and the volume of water likely to enter the reservoir flow as a result of a particular storm flood. Water-level forecasts relating to large, slow-rising rivers can be estimated easily by measuring the water level of upstream stations. Therefore, the data input in such cases will be the water level at two or more stations on the main river or its tributaries. However, for small, flashy rivers, apart from the observations of water level and discharge at relatively small intervals, the use of rainfall data practically becomes unavoidable.

Various considerations, discussed in subsequent sections, go into deciding what type of forecasting model should be used. Input data requirements for calibration and operational forecasts vary significantly from model to model. For example, in case of a simple gauge-to-gauge co-relation model which may be suitable for large rivers, the only data requirement may be water level at two or more stations. However, the use of a suitable comprehensive catchment model requires a number of other data.

Although the accuracy of the forecasts is of primary concern, the constraints in respect of economy and the relative importance and purpose of forecasting may permit a lesser degree of accuracy. In such situations a model may be selected where data requirement may be less rigorous. However, for forecasts at critical sites, such as those located near densely populated areas or otherwise highly sensitive areas, greater accuracy is essential.

Apart from the type of data to be used for forecasting purposes, the information regarding data frequency, the length of record of data and data quality are equally important, and should be duly accounted for in any flood forecasting system planning. Care must be taken to ensure that there is no bias between the data used to develop the forecast procedure or to calibrate models and data used for operational forecasting.

On the whole, the availability and quality of data needed to produce a forecast is improving. The number of automated gages and radars is increasing, while the quality of new satellites and rainfall estimation algorithms is producing enhanced inputs to hydrological forecasting procedures and forecast systems. A key issue in achieving data reliability in hydrological forecasts is the maintenance of the data platforms and the communications system.

7.2.2 Data required to establish a forecasting system

Realistic hydrological forecasts cannot be produced without data. Data required for hydrological forecasting, as discussed in the previous sections, can be broadly categorized as:
(a) Physiographic;
(b) Hydrological;
(c) Hydrometeorological.

Data relating to Geographical Information Systems (GISs) are required for both calibration and for visualization of model states and outputs. The data consist of many types of land cover information such as, soils, geology, vegetation and digital elevation model elevations. Hydrological forecasting system performance will depend on the quality and quantity of the historical data and GIS data used to establish parameters.

Hydrological data relating to river water levels, such as discharges, ground water level, water quality and sediment load, and hydrometeorological data dealing with evaporation, temperature, humidity, rainfall and other forms of precipitation, such as snow and hail, are key to hydrological forecasting. Some or all of the above data may be needed either for model development or for operational use, depending on the model. Over the past ten years, databases and database-processing software have been coupled with hydrological models to produce hydrological forecasting systems which utilize hydrological and or meteorological data and process the data to be used by hydrological models. The latter then produce outputs used by the hydrologist to forecast river flow conditions, including floods and droughts.

An adequate hydrometeorological network is the main requirement for flood forecasting. In most cases, the operational performance of the data network is the weakest link within an integrated system. In particular, for the forecasting of floods and droughts, there needs to be at least adequate precipitation and streamflow/stream-gauge data. If snowmelt is a factor, measurements of snow-water equivalent, extent of snow cover and air temperature are also important. Therefore, when establishing a hydrological forecasting system, it is important to ask the following questions:
(a) Are the rainfall and stream-gauge networks satisfactory for sampling the intensity and

spatial distribution of rainfall and the streamflow response for the river basin?
(b) Are the stream gauges operating properly, and are they providing accurate data on the water level and streamflow?
(c) Are the data communicated reliably between the gauge sites and the forecast centre?
(d) How often are observations taken, and how long does it take for them to be transmitted to the forecast centre?
(e) Are data available to users who need the information for decision-making?
(f) Are the data archived for future use?
(g) Are the data collected according to known standards, and is the equipment properly maintained and calibrated and the data quality controlled?

Analysing the existing network is the first step. An inventory of available monitoring locations, parameters, sensors, recorders, telemetry equipment and other related data should be made and presented in graphical form. In low relief basins, monitoring sites from adjacent basins should also be listed, as data from those sites can be very useful. An evaluation should be performed to identify sub-basins that are hydrologically or meteorologically similar. The main objective is to take advantage of existing hydrometeorological networks operated by various government agencies and the private sector that are relevant for the basin. In some respects, it is preferable that the network serve many purposes, as this may lead to broader financial support of the network.

The sufficiency of networks can be determined according to forecasting needs, and required modifications should be noted. These could include new stream gauges, raingauges and other sensors in the headwaters, or additional telemetry equipment. In some cases, network sites may not be well suited for obtaining flow measurements or other data under extreme conditions. Structural alterations may be required. Interagency agreements may be needed for the maintenance and operation of the network.

There are many sources of such data, ranging from in situ manual observations to automated data collection platforms and remote-sensing systems. Automated data systems consist of meteorological and hydrological sensors, a radio transmitter or computer – data logger – and a downlink or receiver site that receives and processes data for applications. There are many types of automated hydrometeorological data systems that utilize line-of-sight, satellite or meteor-burst communications technology. The rapid transmission of hydrometeorological information is extremely useful to water stakeholders and users because it can be accessed instantaneously by many users by downlinks and/or the Internet. Radar is a very popular and powerful, yet expensive, tool that can be used to estimate precipitation over large areas. The use of geostationary and polar orbiting satellites to derive large volumes of meteorological and hydrological products is advancing rapidly. Remotely sensed data can now be used to provide estimates of precipitation, snow pack extent, vegetation type, land use, evapotranspiration and soil moisture, and to delineate inundated areas.

For information on the instruments to be used in collecting, processing, storing and distributing hydrological and related data, see Volume I of this Guide.

7.2.3 Data required for operational purposes

The basic parameters that control hydrological processes and runoff are initial conditions and future factors. Initial conditions are conditions existing at the time the forecast is made and which can be computed or estimated on the basis of current and past hydrometeorological data. Future factors are those which influence the hydrological forecast after the current time. It can be claimed that the most severe resource management issues exist under extreme conditions: in time, during floods and droughts; and in space, in arid, semi-arid and tropical areas and in coastal areas.

A key variable to be established is the time step needed to adequately forecast a flood for a given location. If the time step is six hours, for example, the data must be collected every three hours or even more frequently. In many cases, supplementing a manual observer network with some automated gauges may provide an adequate operational network. The use of more and more data may become necessary to improve the model efficiency, which will most likely increase the costs. This is a major factor governing the choice for observation, collection and analysis of data to be used for development of a suitable model and for operational flood forecasting. More data entail more expenditure and more time in collection and analysis and man power, for example. Cost-effectiveness of the model vis-à-vis relative accuracy and consequences resulting therefrom should be duly considered when determining the data requirements.

Remote-sensing plays an important role in collecting up-to-date information and data in both the spatial and temporal domains. The use of remote-sensing techniques is vital in areal estimation, in particular of precipitation and soil moisture. Such techniques enhance seasonal forecasting capabilities; contribute to the development of storm-surge forecasting, drought and low-flow forecasting; and help improve risk management.

The rapid spread of the Internet throughout the world has not only produced an excellent mechanism to distribute hydrological data and forecasts to a diverse user community, but has also produced a rich source of data, forecasts and information of use to National Meteorological and Hydrological Services. The Internet provides a source of valuable information for hydrological forecast services. This may include meteorological and hydrological models, hydrological forecasting documentation, geographical information system data, real-time global meteorological forecasting products, hydrometeorological data and hydrological forecasting information. A vast amount of data, software and documents are available for use, and these sources are growing daily. Some sample URLs, or uniform resource locators, are provided at the end of the chapter for reference.

7.3 FORECASTING TECHNIQUES [HOMS J04, J10, J15, J80]

7.3.1 Requirements for flood forecasting models

Given the recognized variability of climate and its expected influence on the severity, frequency and impact of floods and droughts, the importance of forecasting has increased in recent years. This section descibes the basic mathematical and hydrological techniques forming the component parts of any forecasting system. A brief discussion of the criteria for selecting the methods and determining the parameters is also provided. Examples of the use of these components for particular applications are given in 7.4 to 7.6.

Flood forecasting operations are centred around time and the degree of accuracy of the forecast. In fact, a professional assigned with formulating a forecast has to race against time. Clearly, the models to be used by forecasting organizations must be reliable, simple and capable of providing sufficient warning time and a desired degree of accuracy. Model selection depends on the following factors: amount of data available; complexity of the hydrological processes to be modelled; reliability, accuracy and lead time required; type and frequency of floods that occur; and user requirements.

A comprehensive model involving very detailed functions which may provide increased warning time and greater degree of accuracy may have very elaborate input data requirements. All input data for a specific model may not be available on a real-time basis. Therefore, from a practical point of view, a flood forecasting model should satisfy the following criteria:

(a) Provide reliable forecasts with sufficient warning time;
(b) Have a reasonable degree of accuracy;
(c) Meet data requirements within available data and financial means, both for calibration and for operational use;
(d) Feature easy-to-understand functions;
(e) Be simple enough to be operated by operational staff with moderate training.

Indeed, the choice should never be restricted to a specific model. It is always desirable to select and calibrate as many models as possible with a detailed note on suitability of each of the models under different conditions. These models should be applied according to the conditions under which they are to be operated.

Comprehensive models, which are rather complicated, generally require computational facilities such as computers of a suitable size. At many places, however, such facilities are not available. Sometimes suitably trained staff are not available; what is more, these machines cannot be operated because of recurring problems such as electricity failures. Therefore, both computer-based comprehensive models and simple types of model can be developed. A computer-based technique can be used in general, and in case of emergency, conventional techniques, which are generally of a simple type, may be adopted.

Apart from the selection of different models, it is desirable to have a calibration of the models under different conditions. For example, a model may be calibrated with a suitably large data network; however, at the same time, a model must be calibrated for a smaller network and give due consideration to the possible failures in observation and real-time transmission of some of the data. This will be helpful in utilizing the model even in emergency situations in which the data are not available from all the stations. This will require different sets

of parameters to be adopted under different conditions.

7.3.2 Flood forecasting methods

On the basis of the analytical approach used to develop a forecasting model, flood forecasting methods can be classified as follows:
(a) Methods based on a statistical approach;
(b) Methods based on a mechanism of flood formation and propagation.

Forecasting methods in the form of mathematical relationships produced with the help of historical data and statistical analysis have been widely used in the past. These include simple gauge-to-gauge relationships, gauge-to-gauge relationships with some additional parameters and rainfall–peak stage relationships. These relationships can be easily developed and are most commonly used as a starting point while establishing a flood forecasting system. The use of artificial neural networks to forecast flood flows is another modelling approach that has recently gained popularity.

Increasingly, forecast procedures are based on more complete physical descriptions of fundamental hydrological and hydraulic processes. In many instances when forecast flows and stages are needed along rivers, hydrologists use rainfall-runoff models coupled with river-routing models. If precipitation is in the form of snow, snowmelt models are applied. These models vary in accuracy and complexity, ranging from simple antecedent index models to multi-parameter conceptual or process models. With advances in computing and telemetry developments, forecasting models are now more flexible in providing information and allowing new data and experience to be incorporated in real time.

There are many varieties of these basic categories of models, and most differ according to how hydrological processes are parameterized. Models can range from simple ones featuring a statistical rainfall-runoff relationship combined with a routing equation to others characterized by a much higher degree of complexity.

Hydrological models can be classified as lumped, semi-distributed or distributed. Models are either event driven or continuous. If a model is capable of estimating only a particular event, for example the peak flood resulting from a storm, it is known as as event driven. A continuous model is capable of predicting the complete flood hydrograph at a specified time interval. Model selection requirements include the following factors:
(a) Forecast objectives and requirements;
(b) Degree of accuracy required;
(c) Data availability;
(d) Availability of operational facilities;
(e) Availability of trained personnel for development of the model and its operational use;
(f) Upgradeability of the model.

Significant progress over the past two decades has been made in improving the science and performance of such models. However, performance usually varies according to the type of river basin characteristics being modelled, the availability of data to calibrate models and the experience and understanding of the model by the operational hydrologist. There are a large number of public domain and proprietary models available for use in flood forecasting. Chapter 6 of this Guide reviews a wide range of currently available hydrological models.

7.3.2.1 Statistical method

The correlation coefficient measures the linear association between two variables and is a widely used mathematical tool at the root of many hydrological analyses. Regression is an extension of the correlation concept that provides formulae for deriving a variable of interest, for example, seasonal low flow, from one or more currently available observations, such as maximum winter groundwater level (see Draper and Smith, 1966).

The formula for calculating the correlation coefficient r between n pairs of values of x and y is as follows:

$$r = \frac{\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_{i=1}^{n}(x_i - \bar{x})^2 \sum_{i=1}^{n}(y_i - \bar{y})^2}} \quad (7.1)$$

where $\bar{x} = \frac{1}{n}\sum_{i=1}^{n} x_i$ and $\bar{y} = \frac{1}{n}\sum_{i=1}^{n} y_i$

A lack of correlation does not imply lack of association, because r measures only linear association and, for example, a strict curvilinear relationship would not necessarily be reflected in a high value of r. Conversely, correlation between two variables does not mean that they are causatively connected. A simple scatter diagram between two variables of interest amounts to a graphical correlation and is the basis of the crest-stage forecast technique (see 7.3.4 for verification of forecasts).

If either x or y has a time-series structure, especially a trend, steps should be taken to remove this

structure before correlating, and caution should be exercised in interpreting its significance. Time-series techniques may be applied (see 7.5.3) when previous values of a variable such as river discharge are used to forecast the value of the same variable at some future time.

Likewise, regression equations have many applications in hydrology. Their general form is as follows:

$$Y = b_o + b_1X_1 + b_2X_2 + b_3X_3 + ... \quad (7.2)$$

where X refers to currently observed variables and Y is a future value of the variable to be forecast. Regression coefficients estimated from observed Y and X values are indicated by b. The X variables may include upstream stage or discharge, rainfall, catchment conditions, temperature or seasonal rainfall. The Y variable may refer to maximum or minimum stage. The multiple-correlation coefficient measures the degree of explanation in the relationship. Another measure of fit, the standard error of estimate, measures the standard deviation of departures from the regression line in the calibration set. The theory is explained in all general statistical texts.

Linear combinations of the variables are sometimes unsatisfactory, and it is necessary to normalize either the X or Y. A powerful transformation method can be used to transform Y to Y_T by the following equations:

$$\begin{array}{lcl} Y_T = (Y^T - 1)/T & \multirow{2}{*}{if} & T \neq 0 \\ Y_T = ln(Y) & & T = 0 \end{array} \quad (7.3)$$

which encompasses power, logarithmic and harmonic transformations on a continuous T scale. A suitable T value can be found by trial and error as that which reduces skewness or graphically by using diagrams such as Figure II.7.4.

Non-linearity can also be accommodated in a regression by using polynomials, for example, by using X_i, X_i^2 or X_i^3. Alternatively, non-linear regression using function-minimization routines offers a simply applied route to fitting parameters of strongly non-linear equations. The selection of a useful subset from a large potential set of explanatory variables calls for considerable judgement and, in particular, careful scrutiny of the residuals, the differences between observed and estimated values in the calibration dataset. The circumstances giving rise to large residuals are often indicative of adjustments that need to be made. Advantage should be taken of computer facilities and graphical displays of residuals to explore a number of alternative combinations. The exclusive use of wholly automatic search and selection procedures, such as stepwise, stagewise, backward and forward selection and optimal subsets should be avoided.

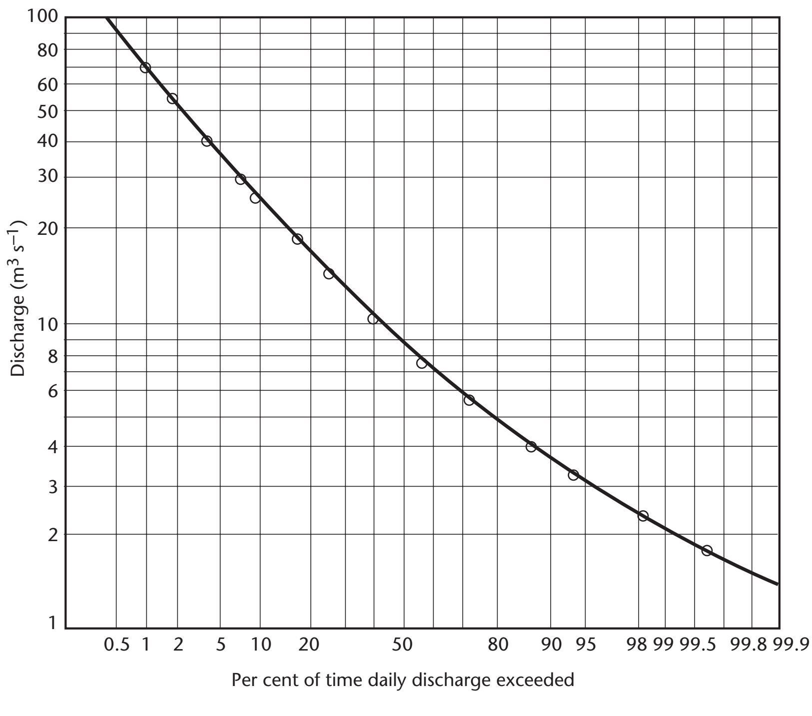


Figure II.7.4. Flow-duration curve of daily discharge

Examples of the application of regression to forecasting problems are given in 7.3.2.3 and 7.4.7.

7.3.2.2 Soil moisture index models

The antecedent precipitation index is described in 6.3.2.2. This method has been a primary tool for operational forecasting in many countries. As a measure of the effect of precipitation occurring prior to the time of the forecast, it provides an index to the moisture in the upper level of the soil. The most frequently encountered indices are the antecedent precipitation index and the antecedent moisture condition. The moisture index methods have two main features with respect to their application to hydrological forecasting. First, because the index is updated daily, it is suited to an event type of analysis rather than continuous modelling. Thus, to apply this method to most forecasting, it is necessary to divide a precipitation period into events or to divide an event into separate precipitation periods. For example, during extended periods of precipitation interrupted by brief periods of little or no rainfall, the decision as to whether one or several storms are involved may be difficult.

The second feature is that the computed surface-runoff volume, when applied to a unit hydrograph, produces a hydrograph of surface runoff only. In order to synthesize the total runoff hydrograph, the base flow must be determined by some other method. The technique is of operational use only if event runoff is of importance and a simple approach is all that can be justified.

7.3.2.3 Simplified stage-forecasting methods

A very common requirement in an event is to forecast the maximum stage or crest. A proven practical technique used in relation to moderate-sized rivers is to construct a simple graphical correlation with an upstream stage hydrograph, thus providing a forecast with a lead time equal to the travel time of the flood wave. Figure II.7.5 illustrates this procedure.

It is common to chain such crest-to-crest forecasts so that the output from an upstream forecast provides the input to a downstream one. Such graphs can often be used to forecast the hydrographs if account is taken of the difference in lag time during the periods of rise and fall. The following correlation relationship is useful when simple station-to-station relationships (Figure II.7.5) are not successful:

$$(h_2)_{t+\Delta t} = f((h_1)_t, I_{loc}) \tag{7.4}$$

where h_1 and h_2 denote maximum stages at an upstream and downstream station, respectively, I_{loc} is the local inflow between the stations, and Δt is lag time. Figure II.7.6 gives an example of the relationship of this type. Sums of discharges at two or more upstream stations at appropriate times, as a combined variable instead of individual tributary stage heights, may reduce the number of variables in the correlation. Variations on these basic approaches can be devised to suit differing circumstances of travel time and tributary inflow. The graphical approach can be replaced by an entirely numerical one by making use of multiple regression (see 7.3.2.1). The regression equation may take the following form:

$$h_{max} = b_0 + b_1Q_1 + b_2Q_2 + \ldots \tag{7.5}$$

where Q_1Q_2 ... are discharges at upstream stations at a given time. Other explanatory variables, such as rainfall and antecedent catchment conditions (7.3.2.2), may supplement or be substituted for discharge.

7.3.2.4 Conceptual streamflow models

There are many basic categories of models, and most vary according to how hydrological processes are conceptualized. Hydrological models and/or forecast procedures use real-time precipitation and

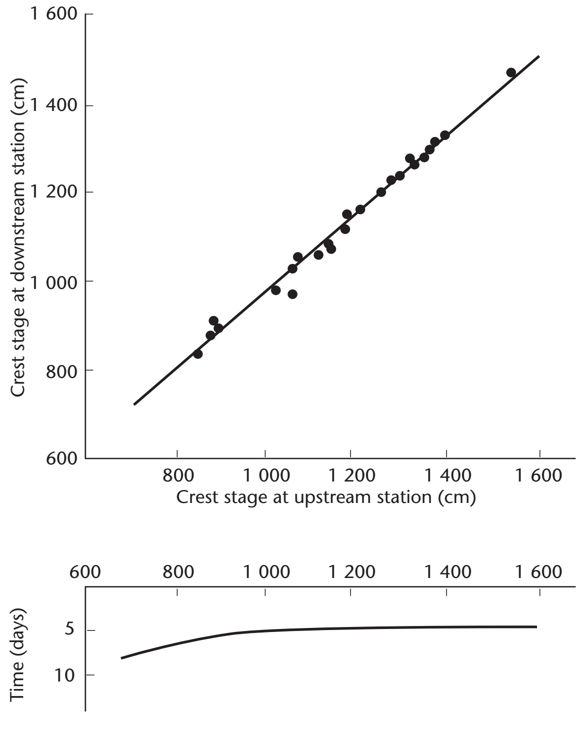
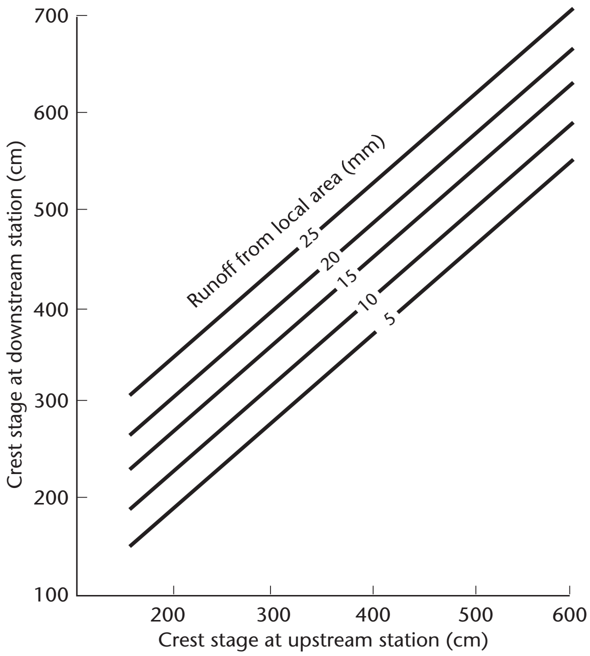


Figure II.7.5. Crest stage and travel time for the Volga river



Note: Average travel time is 24 hours.

Figure II.7.6. Typical gauge relationship with variables for local inflow

streamflow data and translate observed conditions into future stream conditions. Hydrological models or procedures vary in complexity, accuracy and ease of use. Simple hydrological models consist of tables, graphs or empirically derived relationships. More sophisticated hydrological modelling systems use in situ and remotely sensed data, and multiple hydrological models integrated to produce very accurate hydrological forecasts. New developments in personal computer technology have made it possible for complex modelling systems to be run on such computers. These systems are easier to use and sustain than their predecessors.

Large strides have been made over the past two decades in improving the science and performance of models. Model performance varies according to the type of river basin characteristics being modelled, the availability of data to calibrate models and the experience and understanding of the model mechanics of the hydrologist applying the model. Data are usually the limiting factor in attaining acceptable accuracy in operational application. However, with the advances in GIS data availability, hydrological model parameters can be estimated without relying exclusively on historical hydrological data to calibrate the models.

The availability of operational precipitation estimates with high spatial and temporal resolution from weather radars and the substantial increases in computer power have made possible the use of distributed hydrological models. There is a wealth of distributed models, owing to the advent of distributed databases of land-surface and soil characteristics. Carpenter and others (2001), Ogden and others (2001), Beven (2002), and Smith and others (2004*a*) provide recent overviews of distributed hydrological modelling and the issues surrounding its possible use for operational forecasting.

The significant influence of rainfall input uncertainties and model structure and parameter errors on the small scales of flash flood occurrence have hindered the early utilization of distributed models for operational forecasting. Nevertheless, distributed models promise to provide additional information and insight regarding hydrological conditions at locations without existing streamflow observations. In the United States of America, the NOAA-sponsored Distributed Model Intercomparison Project provided a forum to explore the applicability of distributed models using operational quality data and to highlight issues surrounding their use (Smith and others, 2004*b*). To account for uncertainty in rainfall estimates on small scales (see Collier and Krzysztofowicz, 2000) and for hydrological model errors, it is advisable to produce probabilistic, rather than deterministic forecasts in flash-flood-prone areas when using distributed hydrological models. This area of probabilistic flow prediction remains an active research area in hydrology (see Carpenter and Georgakakos, 2004).

7.3.3 **Model updating techniques**

Forecast adjustments are usually based on model output and direct measurements of the state variables. There are many techniques for updating forecasts. If an observation is made of the forecast output Y_i, there is an opportunity to adjust subsequent forecasts with the benefit of the known forecast error $e_i = Y_i - \hat{Y}_i$, where $\hat{Y}_i$ is the forecast estimate. Most adjustments are the result of the subjective judgement of the forecaster, but various mathematical techniques allow this process to be formalized. The underlying principles of the formal approach are described below.

At their simplest, adjustments to forecasts may be made by subtracting the current error from the new forecast. In order to avoid discontinuities, the adjustment is generally blended into the computed hydrograph over several time periods. A more complicated procedure is to subject the error series $e_i, e_2, \ldots, e_i$ to a time-series analysis to extract possible trends or periodicities that can be extrapolated

to estimate the potential new error $\hat{e}_{i+1}$, which can be used to modify the new forecast $\hat{Y}_{i+1}$.

There are two major types of real-time model updating:
(a) Parameter updating, where the estimates of some, and possibly all, of the model parameters are updated regularly on the basis of incoming data such as rainfall and flow. These data are obtained from conventional telemetry or the more modern supervisory control and data acquisition systems, also known by their acronym, SCADA;
(b) State updating, where estimates of the state variables in the model, such as flow or water level, are updated regularly on the basis of incoming data.

Sometimes these updating operations are carried out in a fully integrated manner by using some form of parameter-state estimation algorithm such as the extended Kalman filter. Alternatively, they are carried out concurrently but in separate algorithms. These algorithms are normally known as recursive estimation algorithms because they process data in a recursive manner whereby new estimates are functions of previous estimates, plus a function of the estimated error. Examples of these algorithms are the recursive least squares algorithms, widely used in operational hydrology (see Cluckie and Han, 2000) and the recursive instrumental variables algorithm, as described in Young (1993).

The Kalman filter and the extended Kalman filter are recursive estimation techniques that have been applied to hydrological forecasting, but they require considerable mathematical and hydrological skills to ensure that the forecast model is in a suitable form for analysis.

The generic form of the recursive parameter estimation algorithm is as follows:

Innovations process (one-step-ahead prediction)

$$\hat{a}_t = \hat{a}_{t-1} + G_t\{y_t - \hat{y}_{t|t-1}\};\ \hat{y}_{t|t-1} = f\{\hat{a}_{t-1}, \hat{y}_{t-1}\} \tag{7.6}$$

While the generic form of the state estimation algorithm is:

Model equation

Prediction: $\hat{x}_{t|t-1} = f\{\hat{x}_{t-1}, \hat{a}_{t-1}\}$

Innovations process

Correction: $$\hat{x}_t = \hat{x}_{t|t-1} + G_t\{y_t - \hat{y}_{t|t-1}\} \tag{7.7}$$

where $y = g\{x_t\}$ is the observed data that is related to the state variables of the model in some defined manner and G_t is a time variable matrix, often called the system gain, that is also computed recursively and is a function of the uncertainty in the parameter or state estimates. An algorithm that combines these two recursive estimation operations is often called a data assimilation algorithm (see Young, 1993).

However, a more conceptual technique for adjusting the output of a hydrological model may also be used. The method does not require any changes in the model structure or in the algorithms used in the model. Rather, this approach adjusts the input data and, consequently, the state variables in such a way as to reproduce more closely the current and previous flows. These adjusted values are then used to forecast the hydrograph.

Forecast adjustments need not be based solely on the output of the model. It may also be accomplished by using measurements of state variables for comparison with the values generated by the model. For example, one such technique uses observed measurements of the water equivalent of the snow cover as a means of improving the seasonal water supply forecasts derived from a conceptual model. Direct substitution of field measurements for numerically generated values of the state variables of the model would be incorrect because, in practice, model simplifications can result in such state variables loosing their direct physical identity.

7.3.4 Forecast verification

Forecast verification characterizes the correspondence between a set of forecasts and a corresponding set of observations. No forecast system is complete without verification procedures in place to conduct administrative, scientific and user oriented verification of the forecasts.

A variety of statistics can be computed to evaluate forecast skill. The statistics to be used will depend on the type of forecast and the purpose of the forecast and of the verification. A study of the utility of proposed metrics to effectively characterize the forecast skill should be conducted prior to implementing a verification programme.

To be effective, a verification system must include a forecast archive and the observations against which the forecasts are to be measured. In addition, a baseline forecast must be included to assist with the interpretation of the computed verification

measures. Selection of the baseline forecast will depend on the forecast type to be verified and the forecast process used to develop the forecasts. For short-term deterministic forecasts of less than two days, persistence is a useful baseline.

For longer-term forecasts and for probabilistic forecasts, climatological distributions or lagged climatology are more appropriate baselines. If the forecast process consists of several steps, additional intermediate data must also be archived to enable validation of each step in the forecast process. If possible, the input data used to compute the forecasts should be archived to enable hindcast studies of possible forecast process updates. The data to be archived should include the observations, the input forecasts, such as precipitation and temperature, and the model parameters, including rating curves. Joliffe and Stephenson (2003) are an excellent reference, providing more detailed information. In 1995, WMO developed MOFFS, the management overview of flood forecasting systems, to seek an internationally applicable basis for providing fast, focused information on the performance of flood forecasting systems based on exceedence of specified trigger levels on rivers. The objective of MOFFS is to swiftly identify and highlight deficiencies in the facilities and performance of individual flood forecasting systems in order that appropriate management action may be taken to remedy the defects before the next flood event occurs.

7.4 FORECASTING FLASH FLOODS [HOMS J04, J10, J15]

Flash floods are rapidly rising flood waters that are the result of excessive rainfall or dam break events. Rain-induced flash floods are excessive water flow events that develop within a few hours – typically less than six hours – of the causative rainfall event, usually in mountainous areas or in areas with extensive impervious surfaces such as urban areas. Although most of the flash floods observed are rain induced, breaks of natural or human-made dams can also cause the release of excessive volumes of stored water in a short period of time with catastrophic consequences downstream. Examples are the break of ice jams or temporary debris dams.

7.4.1 National flash flood programmes

Prior to the advent and availability of high-resolution spatially extensive digital data from weather radars and from satellite platforms, and of high-resolution digital terrain elevation data, forecasting of flash floods, as well as with the required spatio-temporal resolution, was not possible on a national scale. In recent years, however, high-resolution data have become available in most countries, and expanded computer capabilities have made it possible to develop national flash flood forecasting programmes.

7.4.1.1 Cooperation between hydrologists and meteorologists

Owing to the short concentration times of flash floods, the timely and accurate detection and short-term prediction of rainfall and streamflow and/or water levels are important ingredients of a successful flash flood forecast and warning system. This renders flash flood forecasting a truly hydrometeorological endeavour, which benefits much from close collaboration between meteorologists and hydrologists in national and regional forecasting centres. In addition, the local nature of rain-induced flash floods requires detailed regional and local observations, understanding and modelling of the heavy rainfall and the runoff-production/channel-routing processes in the flash-flood-prone areas, supported by databases of high resolution in both space and time.

7.4.1.2 Cooperation between national and regional or local agencies

Even when national flash flood forecasting programmes are in place, regional and local involvement is necessary for the operation of the systems to succeed. Individual regional and local physical settings significantly affect flash flood genesis and development. The meteorological and hydrological situation may change from the time of data input at the national level to the time when regional and local response to forecasts is required. The error levels in the measurements by weather radar and satellite data vary considerably from place to place. Finally, individual end-users at the local level – the public at large, individual industries, water resources management agencies and so forth – are likely to have different requirements for flash flood warnings that may not all be addressed by the national flash flood forecast programme. This national and regional or local collaboration ideally involves regional forecast offices, local response agencies and end-users.

It may be necessary for end-users to develop additional products that utilize the national flash flood forecasts and other ancillary information produced by the national forecast centres to address their

individual needs at the local level. For instance, this may include procedures for further refinement of the forecasts for certain flood-stage levels not addressed by the national products, or installation and operation of local automated networks of raingauges and special-purpose radars in areas where national weather radars and satellites do not provide reliable data. In such cases, the national flash flood programme provides flash flood guidance.

7.4.1.3 Cooperation with end-users

For flash flood forecasts that are highly resolved in space and time, it is desirable to establish a significant forecaster-user collaboration programme that will serve several purposes: inform the users – the regional weather service offices, the local response agencies, the public at large or other end-users – as to what the national flash flood forecasts mean; provide information about forecast validation and the limitation of the national systems implemented; support decision-making at the local level; develop guidelines for appropriate user action when warnings are issued; identify ways to receive feedback from the end-users as to the performance of the operational system; and other purposes. This collaborative programme will in the long term help improve the local effectiveness of national flash flood forecast products.

In several countries, flash flood forecasts are disseminated by means of watches and warnings. If meteorological conditions conducive to heavy rainfall are observed or foreseen for an area, a watch is issued on radio and/or television. This alerts residents in the area to the potential occurrence of rainfall that could produce flooding. When flood-producing rainfall is reported, the watch is followed by a warning advising the residents in the area to take necessary precautions against flooding.

7.4.2 Local flash flood systems

There is a wide variety of flash flood forecasting and warning approaches implemented for specific gauged sites. They range from self-help procedures based on local networks of automated stream gauges to more sophisticated procedures that include local short-term rainfall and flow forecasting. These procedures are designed to provide early warning for local communities, utility companies and other regional or local organizations so that they can act immediately on receiving the warning. A few representative site-specific approaches are discussed below.

7.4.2.1 Self-help forecast programmes

Self-help flash flood warning systems are operated by the local community to minimize delays in the collection of data and dissemination of forecasts. A local flood warning coordinator is trained to prepare flash flood warnings based on pre-planned procedures or models prepared by qualified forecast authorities. The procedures are employed when real-time data and/or forecast rainfall indicate a potential for flooding. Multiple regression equations provide an operationally simple flash flood forecasting technique that is summarized in a simple flood advisory table. The procedure is suitable for a range of different flood-producing conditions of rainfall, soil moisture and temperature.

The growing availability of microprocessors has led to an increased tendency to automate much of the data collection and processing that are required to produce flash flood warnings. Automatic rainfall and stage sensors can be telemetered directly to the computer that will monitor the data-collection system, compute flood potential or a flood forecast, and even raise an alarm. The most critical component in the self-help system is maintenance of active community participation in the planning and operation of the system.

7.4.2.2 Alarm systems

A flash-flood alarm system is an automated version of the self-help type of warning programme. A stage sensor is installed upstream of a forecast area and is linked by land or radio telemetry to a reception point in the community such as a fire or police station that is staffed around the clock. This reception point contains an audible and visual internal alarm and relay contacts for operating an external alarm. The alarm is activated when the water level at the sensor reaches a pre-set critical height.

7.4.2.3 Integrated hydrometeorological systems

These systems are more sophisticated state-of-the-art systems and are generally implemented by utilities and other regional or local organizations that maintain in-house hydrometeorological expertise. In most cases these systems provide the most reliable flash flood forecasts for specific locations. Typical implementations involve integrated hydrometeorological models, either conceptual or process based (see Georgakakos, 2002). The components of these models consist of a regional interpolator of operational numerical weather

prediction information to the scale of analysis, 100 km^2 or less, a soil-water accounting model and a channel-routing model. To account for uncertainties in real-time numerical weather predictions and sensor-data configurations, state estimators or assimilators provide feedback to model states from available real-time observations. Various forms of the extended Kalman filter and non-linear filters have been used in these systems.

An example of the implementation and use of integrated hydrometeorological systems is the Panama Canal watershed flash flood forecast system: more information may be found in Georgakakos and Sperfslage (2004). The 3 300-km^2 Canal watershed has been subdivided into 11 sub-catchments based on topography, stream gauge availability, reservoir location and local hydrometeorology (see Figure II.7.7). Short-term forecasts covering one- to six-hour periods are necessary to mitigate damage to Canal equipment and operations. A meteorologist and a hydrologist operate the system and interpret the rainfall and flow forecasts.

There is a 10-cm weather radar and more than 35 automated ALERT-type raingauges in the region. The computational grid of the US National Weather Service operational numerical weather prediction model ETA covers the region with an 80-km resolution and provides large-area forecasts of atmospheric state twice daily with six-hourly resolution and a maximum lead time of several days.

The rainfall forecast component uses information from the 80-km ETA model and upper-air radiosonde and surface meteorological data. The precipitation model produces sub-catchment rainfall forecasts that are compared to the merged radar gauge estimates to produce a forecast error. These rainfall forecasts are fed into the soil water accounting model of each sub-catchment that generates runoff and feeds the channel-routing model. A separate state estimator is used to update the soil water model states from real-time discharge observations.

An important aspect of local hydrometeorological systems is forecast validation for significant flash flood events. This activity provides useful information to forecasters to assist them with the interpretation of the system forecasts and the translation of these forecasts into warnings and watches. Typical least squares performance measures may be used, such as residual mean, residual variance, mean square error and coefficient of efficiency, together with other measures of performance

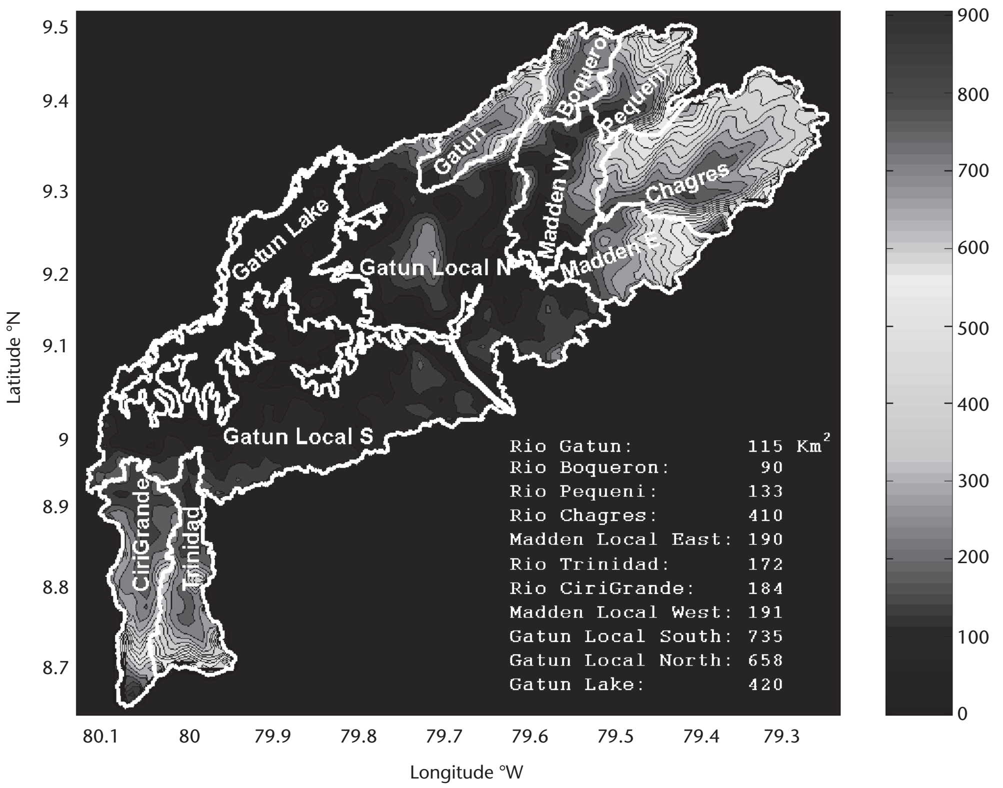


Figure II.7.7. The Panama Canal watershed showing terrain elevation (1 km digital terrain model) and sub-catchments (Georgakakos and Sperfslage (2004))

produced in collaboration with the forecast users, including errors in forecast water volume over a given duration, peak hourly flow timing and magnitude. Figure II.7.8 is an example of a flash flood warning.

7.4.3 Wide-area flash flood forecasts

The ability to measure precipitation routinely with high spatial and temporal resolution and the availability of high-resolution spatial databases for the land surface and subsurface have led to the emergence of flash-flood-scale, operational, wide-area forecasts produced by national agencies. Two main approaches may be identified for the production of wide-area flash flood forecasts with high resolution: (a) those that use the concept of flash flood guidance and (b) those that are based on spatially distributed hydrological models. In either case, rainfall observations and forecasts highly resolved in space and time are necessary.

To obtain rainfall estimates on the scales required for flash flood forecasting, dense raingauge networks are needed. For national flash flood forecasting over large areas with high resolution, rainfall estimation on such small scales includes data from automated raingauges complemented by data from regional weather radars and/or satellite sensors. Different sensors measure different attributes of rainfall and a merged product is often computed as a best estimate that combines all available data. It is often useful to produce measures of uncertainty in the rainfall estimates because measurement errors vary from sensor to sensor and region to region.

Many studies discuss operational quantitative rainfall estimation achieved by merging raingauge and weather radar data, ranging from the early results of Collinge and Kirby (1987) in the United Kingdom of Great Britain and Northern Ireland to more recent results reported in the United States by Fulton and others (1998) and Seo and Breidenbach (2002). In such cases, the spatial variability of the rainfall field on flash flood occurrence scales is obtained mainly from weather radar data, while use of the automated raingauges is made to correct field-mean or range-dependent bias of the weather radar estimates using a variety of procedures, as described, for example, by Cluckie and Collier (1991), Braga and Massambani (1997) and Tachikawa and others (2003).

Satellite rainfall data are often calibrated with weather radar data existing in similar hydroclimatic regions and/or any sparse local or regional automated raingauge networks. Combinations of polar-orbiting and geostationary satellite products are also under development (see Bellerby and others, 2001).

7.4.4 Flash flood guidance

The concept of flash flood guidance has been used for wide area forecasts of flash flood occurrence since the mid 1970s in the United States (Mogil and others, 1978). Flash flood guidance is the volume of rainfall of a given duration, for example, one to six hours, over a given small catchment that suffices to cause minor flooding at the outlet of the draining stream. The volume estimate is updated frequently and is used to assess the potential for flooding when compared with volumes of observed or forecast rainfall of the same duration and over the same small catchment.

Determination of flash flood guidance in an operational environment requires the development of the following tools:

(a) Estimates of threshold runoff volume of various durations, done offline;
(b) A soil moisture accounting model to establish the curves that relate threshold runoff to flash flood guidance for various estimated soil moisture deficits (Sweeney, 1992).

Early flash flood guidance operations used statistical relationships to develop the required threshold runoff estimates from a variety of regional and local data, such as topographic and climate data. Using existing digital spatial databases of land-surface properties such as terrain, streams and land use or land cover, together with GIS, Carpenter and others (1999) set the threshold runoff estimation problem on a physical basis and provide methods for developing objective threshold runoff estimates on a national scale with high resolution. For a given small catchment, the basic threshold runoff relationship is as follows:

$$Q_{flood} = Q_p(R, t_r) \qquad (7.8)$$

where Q_{flood} is the flow that is considered likely to cause minor flooding at the catchment outlet, and Q_p is the peak of the surface runoff over the catchment caused by the effective rainfall volume R, the threshold runoff, of the given duration, t_r. The Q_{flood} may be estimated by the flow of a given return period, for example, two or four years, or by hydraulic formulae for uniform steady flow at stream bankfull and at the catchment outlet. Synthetic or geomorphologic unit hydrograph formulations may be used to estimate Q_p from R and t_r. The use of bankfull flow and geomorphologic unit hydrograph

Figure II.7.8. Flash flood warning sign

formulations requires no calibration and produces threshold runoff estimates that are conservative in terms of flood damage.

Channel cross-sectional properties at the catchment outlet are required in order to estimate bankfull flow and the geomorphologic unit hydrograph runoff peak. Such estimates are typically obtained from regional regressions of channel cross-sectional properties, that is, bankfull width, or hydraulic depth that use GIS-estimated catchment properties – area, stream length and average stream slope – as predictors. These regional regressions are based on data obtained from surveys of natural streams in the region of interest and are used to provide estimates of channel cross-sectional parameters in all of the small catchments of the region.

The resolution of the digital terrain elevation data limits the size of the smallest catchments for which threshold runoff analysis may be made. For instance, a 90-m resolution terrain database may be used to produce catchment properties, such as area and stream location, length and slope, with relative errors between +/–10 per cent and +/–25 per cent for catchments greater than 5 km^2. For such catchments, up to a maximum size of 50 km^2, typical errors of threshold runoff estimates based on GIS analysis of digital terrain elevation data can reach +/–30 per cent of the value estimated at sites with a full complement of hydrometeorological data.

Threshold runoff is the volume of effective rainfall of a given duration generated over a small catchment that is sufficient to cause minor flooding at the outlet of the draining stream. Once the threshold runoff estimates have been obtained for the regions of interest or for the entire country, they are used in conjunction with real-time estimates of soil water deficit to produce threshold runoff. The procedure is outlined below (see Georgakakos, 2004).

Typically, the national forecast services run a hydrological model operationally over basins of area in the order of 1 000 km^2 to produce streamflow estimates and forecasts at each of several forecast preparation times. Upon completion of these operational forecast runs, the current estimates of the soil water indices, valid at the forecast preparation time, are stored. To support flash flood computation under these initial conditions, the hydrological model is run offline in “what if” scenario runs with increasing amounts of rainfall volumes of the same given duration. These “what if” runs use the same initial soil water estimates produced by the model during a normal operational run. The surface runoff volume produced by these runs is plotted against the volume of required rainfall of a given duration. This plot may then be interpreted as the relationship of threshold runoff (effective rainfall or surface runoff volume) to flash flood guidance (actual rainfall volume). It is used with the estimated value of threshold runoff for the catchment to obtain the required flash flood guidance volume, both of the same given duration.

Estimates of most recent catchment rainfall volume of duration equal to the flash flood guidance duration may then be used to determine whether a flash flood is imminent in a certain catchment. Likely flash flooding occurrence is indicated when the observed rainfall volume is greater than the flash flood guidance estimate. Following this procedure, maps of entire regions highlighting catchments with a high potential for flash flooding may be produced on regional and national scales. Similar maps may be produced showing the future potential of catchments for flash flooding using forecast, rather than observed, catchment rainfall volumes of a given duration. The US National Weather Service uses a national operational implementation of flash flood guidance for forecasts of wide area flash floods. A regional implementation of flash flood guidance is operated for the countries of Central America. National programmes for collecting the necessary flash flood occurrence data to validate the flash flood forecasts produced on the basis of flash flood guidance are a necessary complement to the operational forecast programmes.

7.4.5 Dam-break flash flood forecasting

Catastrophic flash flooding results when a dam fails and the outflow passes through the breach in the dam and inundates the downstream valley. Methods used to predict the floods that result from such failures are described in 6.3.5.4.

In recent years, the development of GIS and digital terrain elevation data of high resolution has led to the production of risk maps for specific areas downstream of existing dams. These inundation maps indexed with flood wave travel time information are useful when distributed to local officials downstream of a dam site for the development of contingency evacuation plans.

7.4.6 Storm surges in rivers

Storm surges in the open sea are produced by wind and atmospheric pressure and can generate gravity waves that propagate upstream into rivers. Suitable

techniques to forecast the development and propagation of the storm surge in the open sea, such as the US National Weather Service SPLASH model (Jelesnianski, 1974) and its propagation into bays – as presented by Overland (1975) – are required to define the surge at the river mouth, where it is then routed upstream via a suitable dynamic-routing technique. As the upstream movement of the gravity wave is opposed by the downstream flow, routing of the storm surge upstream may best be accomplished by dynamic-routing techniques (see 6.3.5). Hydrological routing techniques or kinematic-hydraulic routing techniques are not suited to prediction of wave motions that propagate upstream. Also, the inertial components of the gravity wave that are ignored in the diffusion-hydraulic routing techniques are too important to be neglected in the case of a storm surge. A number of papers on tidal rivers have been published by the United Nations Educational, Scientific and Cultural Organization (1991). More recent applications involve the use of GIS to produce risk maps for areas prone to flooding by combined storm surge and flood waves (see publications of the WMO Tropical Cyclone Programme).

7.4.7 Urban flooding

Continued urbanization of natural flood plains has contributed to a sharp increase in loss of life and damage to property. Rapid demographic and social changes, coupled with increasing land prices and environmental concerns relating to water pollution and potential climate change characterized by increased variability and extreme magnitude, make advances in urban water management worldwide all the more urgent (Pielke and Downton, 2000; Dabberdt and others, 2000).

There are two types of urban flooding. First, urban areas can be inundated by rivers overflowing their banks – this is fluvial flooding. Areas of inundation are forecast from the specific river-stage forecasts. Second, urban flooding can occur in local drainage as a special case of flash flooding.

A considerable volume of literature has been published on urban hydrology and water management, for example, reviews in Urbonas and Roesner, 1993; Kovar and Nachtnebel, 1996; and Dabberdt and others, 2000. Unique characteristics of urban hydrology are large areas of impervious or near-impervious areas and the co-existence of both natural and technological drainage systems: sewers, levees, pumps, detention basins and the like. As a result, surface runoff production from rainfall is highly variable and non-homogeneous, and the flow of water and contaminants is accelerated toward higher peaks of outlet hydrographs. High spatial-temporal variability in rainfall translates into high spatial-temporal variability in runoff, as the urban catchments do not significantly dampen such fluctuations. The technological drainage and improvements to the natural drainage make for earlier and higher peak flows. With respect to hydrological impacts, the flood prediction and control problem becomes severe for events with 5 to 100 years' return period, while the water quality problem can be acute with storms occurring with short return periods of even less than two years.

Owing to the characteristics of the urban response to rainfall and contaminant forcing, very high spatial and temporal resolutions are required in data, models and controls over large urban areas in order to ensure effective water resources management (Dabberdt and others, 2000). Thus, digital terrain elevation data, distributed hydrological models and weather radar data – combined with in situ automated raingauge data and GIS – can be used to develop urban runoff forecast and management systems. (Cluckie and Collier, 1991; Braga and Massambani, 1997; Georgakakos and Krajewski, 2000; Kovar and Nachtnebel, 1996; Riccardi and others 1997). In areas where significant urban growth is combined with mountainous terrain and convective weather regimes (Kuo, 1993), there is a great need to develop urban water resources management systems capable of very high resolution over large urban areas.

7.4.8 Flooding from local drainage

In this case, intense rainfall over the urban area may cause flash flooding of streets and property in low-lying areas or built-up areas in old waterways, underpasses and depressions in highways. Such floods arise primarily from inadequate storm-drainage facilities, and are invariably aggravated by debris clogging inlets to pipes and channels or outlets of retention basins. Flood warning schemes similar to those outlined for flash floods can be employed. These usually consist of local automated flash flood warning systems or generalized warnings that are based on national flash flood guidance operations. It is also possible to target the flash flood guidance estimates for the urban environment on the basis of very high resolution digital spatial databases of terrain, drainage network, both natural and technological, and existing hydraulic works.

On causeways subject to flooding, traffic can be alerted by using lights activated in the same manner as the flash flood alarm system. Urban flooding

usually affects sewer systems, even when waste water and storm sewerage are piped separately. Forecasts of urban runoff can be helpful in the treatment of sewage and the handling of polluted flood water in combined systems. The opposite problem is the high level of pollution accompanying urban runoff. Since this is ultimately discharged into natural watercourses, it leads to increased pollution with problems for downstream water users. The forecasting of pollution loads depends on forecasting urban flood runoff.

7.5 LONG-TERM FORECASTING

7.5.1 Water supply forecasting

Water supply forecasts are essential for the operation of domestic, industrial, irrigation and hydroelectric water supply systems. Forecasts commonly take the form of flow volumes over specific durations: annual, seasonal or monthly. The duration depends on the nature of the demand and the amount of storage in the system. Since water supply forecasts cover a wider time span than meteorological forecasts, errors will always be inherent because of climatic events during the forecasting period. Therefore, it is recommended that several forecast values with probabilities of exceedance be issued (see 7.3.4).

The choice of the forecasting technique is governed by the character of the drainage basin, available data and user requirements. Water supply forecasts may be made by using three basic techniques:
(a) Snowmelt forecasts;
(b) Conceptual models;
(c) Time-series analysis.

Snowmelt methods are used in basins where snowmelt runoff dominates the flow regime. Forecasting of snowmelt is described in 7.6. Normally, some measures of the snow-water equivalent and the basin losses can be related empirically to total seasonal runoff by regression techniques. Satellite measurements of snow cover have been related to the discharge of the Indus river, for example, and reasonable results have been obtained in this basin, where conventional ground data are very scarce. These methods are primarily suited to forecasts of total runoff volume and do not describe the time distribution of the runoff.

Conceptual models can be used for water supply forecasting by running the model repeatedly and using a number of historical climate time series as inputs. The output becomes a range of forecasted values to which probabilities of exceedance can be assigned. Models used for water supply forecasts should be calibrated so that deviations between observed and simulated runoff volumes are minimized. Since short-term variations are of minor importance, simple model structures may yield satisfactory results.

Time-series methods may be useful for water supply forecasts, where discharge is a valid measure of the state of the basin. The forecast relationships are generally very simple to apply. Regression models in which seasonal runoff is forecast from previous hydrological and climatic variables may be regarded as a special case of time-series methods.

Long-term forecasts, especially of seasonal runoff, are often expressed in probabilistic terms: a statistical distribution of possible runoff volumes is contingent on rainfall subsequent to the date when the forecast is made. One source of uncertainty is the future weather between the date of preparing the forecast and the operative date of the forecast. For example, if a regression-based forecast gives the following equation:

$$Q_{summer} = b_0 + b_1 R_{autumn} + b_2 R_{winter} + b_3 R_{spring} + b_4 R_{summer} \tag{7.9}$$

a less informative, probabilistic forecast can be issued after only receiving the rainfall data for the previous autumn and winter. The probabilistic component must take into account the distribution of possible spring and summer rainfalls that might occur.

Unless the forecast model is very simple, it is almost certain that it will be necessary to simulate possible Q_{summer} values either by repeated sampling from the distribution of R_{spring} and R_{summer} values or by repetitively applying the model to the historical traces of R_{spring} and R_{summer}. If the sampling approach is adopted, it will be necessary to incorporate any correlation that might be present between the independent variables. If the historical approach is used, at least 30 years of record is desirable to obtain a representative range of combinations. The application of this technique is not limited to regression models. Any hydrological forecasting model can be perturbed retrospectively by real or synthetic data to construct a distribution of possible outcomes. A more realistic description of the distribution of actual values is obtained if a noise term is included in the model. This can be accomplished by adding to each forecast a random number whose standard

deviation is equal to the standard error of the model estimate. A more detailed discussion is provided in *Long-range Water-supply Forecasting (*WMO-No. 587, Operational Hydrology Report No. 20).

7.5.2 Flow recession forecasting

Long periods without rain are a feature in many parts of the world, particularly where continental and highly seasonal tropical and subtropical climates prevail. The occurrence of prolonged dry periods is significant for agriculture, which can be adapted to suit conditions by using particular practices, growing crops adapted to the conditions or providing irrigation. Drought takes place when the period without rain extends beyond the normal duration, placing stress on plants and further depleting water resources. It is therefore important form an operational point of view to forecast drought or to provide projections on how long drought conditions will last.

There is no single, simple definition of drought, as its nature will vary with the climate type, and the impact of the drought, for example, water supply, irrigation and stock rearing. Where drought is a regular occurrence, its severity, which is a factor of duration and temperature, becomes important. Drought extremes may result from an early start relative to the normal dry season or a delay in the return of wet conditions, or a combination of both. A simple means of recording drought duration can be defined as follows:

(a) Drought begins after a period of 14 days without rain;
(b) Drought ends after a period of 20 days during which rainfall is recorded on 11 or more days;
(c) As well as duration, intensity of drought can be recorded by cumulative temperature, that is, in degree days.

The Palmer Drought Severity Index (Palmer, 1965) is widely used in the United States as a means of defining drought conditions. The method uses current and recent measurements of temperature and rainfall, which are mathematically related through local mean values to provide an index of severity between –4, very dry, to +4, very wet. The method lends itself very well to mapping and GIS displays and is routinely published on the Web by the Drought Information Center of the National Oceanic and Atmospheric Association (www.drought.gov).

The characteristic behaviour of a river and, catchment to drought can be expressed as a flow-duration curve and a recession curve. A flow-duration curve is clearly a probability relationship taken over the whole of the historic record, and it is therefore possible to equate a current flow to a probability level, and thus project the situation for more extreme flows. A flow-recession curve is constructed by plotting the relationship between flows at set interval separations, for example daily, 5 days or 10 days; the size of interval is influenced by the total length of the dry season and size of catchment. Thus plotting Q at t_0 and t_{-5} throughout the period of declining flow, a curve of the form:

$$Q(t) = Q_0 e^{-C(t-t_0)} \tag{7.10}$$

is produced. Successive years of recession curves can be combined by eye to give a full recession relationship. This allows the current situation to be assessed in terms of the overall catchment recession and to provide an estimate of possible future duration and severity of projected conditions, for example, one or two months ahead.

Meteorological forecasts can be of value in drought management. Most major forecast services now give a long-range forecast for durations of two to six months. These are broad in their approach, and are usually expressed in terms related to average or extreme conditions.

Analysis of the falling limbs of hydrographs, or river recessions, is an important component of flood and low-flow analysis; in forecasting, however, its use is largely confined to low-flow forecasting. Some low-flow forecasting is achieved by analysing master recession curves on the large river basins, thus making it possible to forecast weeks or even months ahead. This type of forecasting is of value to hydropower and irrigation schemes where the long-term supply of water is vital to optimal management practice. In addition, there is a highly specialized area where the principle long-term forecasts are produced by meteorologists using sophisticated global climate models. The subsequent hydrological work then focuses principally on the development of forecasts of flow and aquifer levels for use with reservoir control rules and water allocation strategies.

The most direct method is probably to perform a graphical correlation between the current flow or stage and flow or stage n days ago where n = 1, 2, 4, . . . (see 7.3.2.1). The defined relationship can be used to extrapolate forward in time if there are no disturbing influences, such as precipitation events. Departures from the most characteristic line can often be associated with natural or man-made phenomena, and this

information can also be brought to bear on any particular forecast.

7.5.3 Time-series analysis

A set of observations that measures the variation in time of a particular phenomenon such as the rate of flow in a river or the water level in a well or lake is described as a time series. A time series can be specified in continuous or discrete time, depending on whether observations of a system state variable such as flow are made continuously or quantized into a discrete set of measurements which approximate the variation of the state variable over time (see Kottegoda, 1980).

Since runoff is an indicator of the state of the drainage basin, univariate time-series analysis may be used to establish forecast relationships. One such approach is to use autoregressive moving average models, (Box and Jenkins, 1976) that are well suited for use in basins with limited precipitation data, because only antecedent discharge is needed to make a forecast of this type:

$$Q_{t+1} = a_0 Q_t + a_1 Q_{t-1} + a_i Q_{t-2} + \ldots + b \qquad (7.11)$$

where Q_{t+1} is the forecast with unit lead time and Q_{t-i} are the measured values earlier than i time increments. Coefficients a_i and b are estimated in the time-series analysis. In addition to the forecast value Q_{t+1}, a time-series model can yield the distribution of possible deviations from the forecast value so that an estimate of forecast error is readily available. If a time-series forecast of monthly flows is to be reliable, then autocorrelation in the monthly time series must be large. This is the case in large rivers and in streams draining large aquifers and lakes. As a rule, however, forecasts are feasible only one to four months ahead. It is possible to include meteorological variables in a time-series model but, if such data are available, it is often preferable to make forecasts by using regression or a conceptual model. Time-series models may also be fitted to the error series as discussed in the next section.

7.6 SNOWMELT FORECASTS

7.6.1 General

Many countries use forecast methods based on conceptual models of snowmelt runoff (see 6.3.3). Such methods make it possible to forecast snowmelt from observational and forecast meteorological data. Short- and medium-term forecasts are possible for rivers and lowlands, and medium- and long-term forecasts, for streams in mountainous areas. Seasonal volume forecasts may be prepared for lowland and mountain basins, where snowmelt runoff produces a significant portion of the total streamflow.

Snowmelt runoff is a characteristic feature of the regime of lowland rivers in temperate and cold climates and of some of the world's largest rivers, even in tropical zones. Snowmelt runoff of many rivers accounts for 50–70 per cent of the annual runoff, and in dry regions the corresponding figure may reach 80–90 per cent. Runoff estimates are used in reservoir management and planning for consumptive use, power generation, public works and land development. As a result, a number of snowmelt hydrological models have been developed to predict snowmelt runoff with a focus on capturing or predicting peak flows and volumes for engineering design and reservoir management purposes.

7.6.2 Snowmelt runoff processes in lowland and mountain rivers

During snowmelt, many of the processes that govern runoff in lowland and mountain river basins are similar, for example, snowmelt, water retention of snow, snowmelt inflow to a basin, snowmelt runoff losses, water yield of a basin and time lag of snowmelt runoff to the outlet. However, some of the processes differ in two cases. For example, the year-to-year variation in the snowmelt runoff losses from snow and free water are significantly greater in the plain regions than in mountainous river basins. More importantly, higher relief regions will have very different snow-covered distributions, with elevation playing an important role in the amount, redistribution and sublimation of the snowcover.

The total snowmelt runoff from lowland basins depends on the water equivalent of the snow cover at the time the snow begins to melt – the volume of precipitation occurs after the snow has begun to melt – and on the amount of water lost by infiltration and evaporation over the river basin. The first factor can be determined to some degree by measurement; however, these measurements are highly landscape dependent (see Volume I, Chapter 3, of this Guide). The second factor, the subsequent amount of precipitation and the water losses during the runoff period, must be handled by a forecast procedure, either probabilistically or by assuming climatological average values. The possibility of using numerical weather prediction for short-term

forecasts is becoming a viable option in forecasting meteorological forcing. The third factor, snowmelt runoff loss from the basin, is controlled by the infiltration capacity of the soil and surface-depression storage, including large non-capillary pores in the upper soil layer. Evaporation losses are relatively small and vary little from year to year. Snowpack accumulation and ablation, especially during the spring thaw, are significant inputs into daily hydrological forecasting systems, which in turn, are extremely useful for flood prevention and hydroelectric generation. The measurement and characterization of the distribution of snow within a catchment are critical to the prediction of subsequent melt.

Volume I, Chapter 3, of this Guide states that catchment-based assessments of snow are typically derived from snow surveys and snow courses and, as a rule, recommends that in high relief regions, snow courses be at elevations and exposures where there is little or no melting until peak accumulation is achieved. In mild to low relief regions, these surveys need to represent the average snow conditions within a given catchment, and should be carried out on a variety of landscapes in order to properly depict the natural variability of the landscape.

Infiltration of water into the soil during the snowmelt period is a factor that varies greatly from year to year, depending on the soil conditions. The rate of infiltration into frozen soil and the total amount of water absorbed depend on the soil moisture content prior to freezing, the temperature, the depth of freezing and the soil's physical properties. The size of the area covered by depression storage can be expressed mathematically as distribution functions of the depth of water required to fill these depressions. Such functions are relatively stable characteristics for each river basin.

7.6.3 Short- and medium-term snowmelt runoff forecasts

Short- and medium-term snowmelt runoff forecasts for large river basins may be developed as follows:

(a) Lowland river basins are divided into small, partial basins, which are assumed to be hydrometeorologically homogeneous, each with an area of up to 15 000 km^2, and the river system is divided into sections beginning with the upper reaches;
(b) Mountainous basins are divided into altitude zones. The number of zones depends on the difference in altitude between the head and the mouth of the river system, as well as the variability of hydrometeorological conditions with the zone. In the experience of some hydrologists, the optimum altitude range for such zones is 200 to 400 metres with the number of zones around 20;
(c) The models are calibrated with hydrometeorological data from preceding years;
(d) The forecast flows for the partial basins, or altitude zones for mountain areas, are routed to a downstream forecast point (see 6.3.5).

7.6.4 Long-term snowmelt forecasts

To devise a method for long-term forecasting of snowmelt runoff, it is necessary to establish water-balance relationships. This is preceded by the following tasks:

(a) Determination of the relevant characteristics of the river basin, such as topography, land-cover distribution and the nature of the soils;
(b) Determination of any factors governing the way in which water is absorbed by the soil and retained on the surface of the drainage area;
(c) Definition of the basic factors governing the loss of water in the river basin and the extent to which such factors vary from year to year;
(d) Determination of the role of precipitation occurring after the snowmelt has begun, in relationship to runoff, and of the variability of such precipitation;
(e) Evaluation of the accuracy of data for runoff, snow-water equivalent and precipitation.

Snowmelt runoff forecasts may be improved and extended by including probabilistically representative data and/or quantitative meteorological forecasts for the subsequent snowmelt period.

7.6.4.1 Seasonal snowmelt forecasts for the plains regions

The relationship between total snowmelt runoff Q_n and the snow-water equivalent for plains areas may be expressed theoretically as:

$$Q_n = (w_n - f)\int_o^{w_{n-f}} f(y_d)\,dy_d - \int_o^{w_{n-f}} y_d f(y_d)\,dy_d \quad (7.12)$$

where w_n is the snow-water equivalent and f is the total infiltration during the snowmelt period, both expressed in millimetres. The function $f(y_d)$ is the area distribution function in relation to the depth

of water, (y_d), that is necessary to fill depressions on the river basin surface.

In the absence of infiltration or when its intensity is potentially greater than the rate of snowmelt, equation 7.12 can be simplified as follows:

$$Q_n = w_n \int_o^{w_n} f(y_d)\,dy_d - \int_o^{w_n} y_d f(y_d)\,dy_d \qquad (7.13)$$

In this case, the runoff becomes a function of the snow-water equivalent and the infiltration capacity of the basin.

The amount of water contributing to the seasonal snowmelt runoff is calculated for each year as the sum:

$$W = \bar{w}_n + \bar{P} \qquad (7.14)$$

where $\bar{w}_n$ is the mean snow-water equivalent for the basin at the end of winter and $\bar{P}$ is the mean precipitation during the runoff period, both expressed in millimetres.

The mean snow-water equivalent for the basin may be calculated as either an arithmetic mean or a weighted mean. The arithmetic mean method is used when the number of snow-measuring stations in the basin is sufficiently large and when the spatial distribution of these stations is good. The weighted mean method is used when observation points are unevenly distributed over the area and/or when the distribution of the snow cover is irregular. To calculate the weighted mean of the snow-water equivalent, a map showing snowcover average distribution in the area is drawn.

In regions where a thaw may take place in winter, an ice crust often forms on the ground. If measurements are available, the amount of water contained in such crusts should be added to the snow-water equivalent. Very often, direct determination of soil-moisture conditions throughout the river basin, particularly in winter, is not feasible because of the lack of adequate data. This is the main reason why indirect indices are so common.

In dry steppe regions, the difference between precipitation and evapotranspiration characterizes the potential rate of infiltration. In the humid forest zone where every year the autumn soil-moisture content is equal to, or greater than, field capacity, this difference represents changes in the storage of the basin as a whole. The runoff caused by late autumn precipitation can also be used as an index of the retention capacity of river basins in these regions.

7.6.4.2 Seasonal snowmelt forecasts for mountainous regions

In mountainous areas, there tend to be considerable differences in climate, soil and vegitation because of the range of altitudes. These features determine the nature of the snowmelt runoff and flow regime of the streams. Therefore, the most important characteristic of a mountain basin is its area–elevation distribution. The main sources of runoff are seasonal snow, which accumulates in the mountains during the cold season, and precipitation that occurs during the warm season of the year.

Owing to the long period between the beginning and end of the snowmelt period, long-term forecasts of the seasonal flow of mountain rivers are feasible. The most favourable conditions for such forecasts exist where seasonal snow is the main source of runoff and the amount of summer precipitation is relatively small.

Steep slopes, rocks and an extensive, highly permeable deposit of rough rubble in mountainous basins create conditions in which the water finds its way into channels, mainly through layers of rubble and clefts in the rocks. Under such conditions, water losses do not vary greatly from year to year, and there should be a good relationship between seasonal runoff and the amount of snow in the basin. This relationship can be established empirically if measurements are available for a number of years. However, in practice, determining such relationships is often difficult.

7.7 FORECASTS OF ICE FORMATION AND BREAK-UP
[HOMS J45]

7.7.1 General

Many rivers and lakes in middle latitudes freeze over in winter. The most important ice regime phases for which forecasts are made are as follows:
(a) The first appearance of ice;
(b) The formation of complete ice cover;
(c) The break-up of the ice cover;
(d) The final disappearance of all ice.

The ice regime of rivers is closely related to weather conditions. Thus, the dates of the appearance of floating ice and those of the formation and breaking of the ice cover vary over a wide range from year to year. Ice forecasts are of great practical value

for navigation, but many other users apart from those in inland navigation are interested in these forecasts as well.

Exact relationships for calculating thermal and ice regimes are available; however, their application to ice forecasting is severely limited by the stochastic nature of parameters governing the equations, which vary over the time span between the forecast and the event predicted. This subsection discusses the different ice-regime forecasts that exist and short-term forecasts of ice formation and ice break-up.

Modern approaches to the short-term forecasting of ice phenomena are based on thermal balance (Buzin and others, 1989). For forecasts of autumn ice phenomena, formulae for the thermal balance at the boundary between a unit of surface of water and the adjacent atmosphere are used. Factors include direct heat exchange, solar radiation, turbulent heat and moisture exchange with the atmosphere, effective radiation, inflow of heat from the Earth's surface and groundwater, dissipation of stream energy as heat and the inflow of thermal energy from precipitation which falls on the water surface and from the discharge of industrial and household waste waters. While the role of each of these factors in the thermal balance is different, the most important one is the exchange of heat through the open water surface.

Forecasts of the time when the ice cover breaks up are based on calculations of the tensile strength of the melting snow-ice cover, using formulae for the thermal balance and the derivation of a ratio between the durability of an ice cover and the destructive force at which the ice sheet is fractured. The latter is a function of the stream discharge, its water level and the rate at which they have changed over time.

Methods of modelling the formation and break-up of ice are covered in 6.3.6.3 of the present Guide.

7.7.2 Long-term ice forecasts

The development of methods for long-term forecasting of ice phenomena usually includes the following tasks:

(a) Consideration of the dates of ice formation and break-up on rivers across the area under consideration, for example, average dates, variability of the annual dates and the delineation of regions with uniform ice phenomena, the main mathematical instrument being statistical analyses;
(b) Synoptic analysis of conditions leading to freeze-up or ice break-up, in which the northern hemisphere is divided into typical regions, the main mathematical instrument here being discriminant analyses;
(c) Analysis of the distribution of stores of heat in the surface layers of oceans, such as the Northern Atlantic and the northern Pacific Ocean; identifying the principles areas of interest, stores of heat within the limits of which render the greatest influence on processes leading to the formation and destruction of an ice cover on the rivers, the main mathematical instrument again being discriminant analyses;
(d) Determination of quantitative variables for atmospheric processes and ocean fields, such as expanding meteorological and ocean fields by orthogonal functions;
(e) Use of correlation analyses to determine the relationship between the time of ice occurrence and the variables representing the appropriate meteorological and ocean fields.

7.7.3 Ice jams and methods of forecasting high water levels

Dangerous rises in water level and resulting floods may occur during the formation of an ice cover or ice jam, and as a river's ice cover or ice dam breaks up. Jam floods are especially hazardous because they occur in a cold season and sometimes remain for a long time. This causes sheets of water freeze to form an ice field that can cover populated areas and can be almost impossible to remove. Often a sharp fall in water level occurs below the ice jam, starving intakes of water and interrupting water supplies.

On many rivers, maximum ice dam water levels exceed the highest water levels of spring and summer floods. Ice dams can form quickly and cause very rapid increases in water level, without any significant increase in flow.

Ice jams arise more often in those reaches of rivers where ice cover grows out from each bank towards the middle and in an upsteam direction. The more slowly this process develops, the more ice is brought by the current under the frozen ice cover, causing contraction of the flow channel and raising the water level at the approach of the ice field. Such characteristics of freezing are typical of large rivers flowing towards the poles and rivers flowing out of large lakes and after-bays of hydroelectric power stations.

River discharge during freezing, conditions for heat exchange, including air temperature, and the

position of the ice edge in relation to the cross-section are the primary factors to be taken into account in forecasting water levels from ice dams.

For a number of rivers where dangerous jams are frequently observed, physical–statistical relationships have been developed to account for these factors. The following example relates to the Neva river at Saint Petersburg (Buzin and others, 1989):

$$H_{jam} = 1.29H_{X1} + 0.53L + 0.24H_G - 404 \qquad (7.15)$$

where H_{X1} is the average level of the Ladozhskoye lake in centimetres in November; L is the distance of the ice edge in kilometres from the Gorniy Institute station; H_G is the water level at that station in centimetres.

Interestingly, by knowing only H_{X1}, it is possible to issue good warnings of high water levels from ice jams more than one month in advance and to update these for three- to five-day short-term forecasts using equation 7.15.

A generalized method is available for deriving short-term forecasts of the maximum water level resulting from ice jams at critical locations on a river, including those for which there are no long time series of hydrological observations. Initial data required are the gradient of the given reach of the river, water discharge on the day that ice appears (Q_0), air temperature during the freezing ice over the last few days and the curve $Q = f(H)$ at the free ice channel. For a forecast, it is necessary to define a value of the critical gradient l' given by:

$$l' = 0.0154\, g/c^2\, (1 - e) \qquad (7.16)$$

where g is the acceleration due to gravity; c is the Chezy coefficient; e is the porosity of ice fields estimated according to air temperature, where $e = 0.25$ at $\theta = -10$°C and $e = 0.55$ at $\theta = -2$°C.

If equation 7.16 is applied, the difference between I' and the gradient of the open surface can be used with a special table developed for each hydrological post using the data of hydrological observations to define a conversion factor k_p, the winter dimensionless factor, between winter water discharge and the corresponding open channel discharge. The value of the discharge Q_{kr} is calculated using the following equation:

$$Q_{kr} = Q_0 e^{-k_0 T_{ice}} \qquad (7.17)$$

where k_0 depends on the weather conditions during freezing; T_{ice} is the duration of the ice run in days.

For example the coefficient k_0 for the Amur river can be calculated using the following equation:

$$k_0 = 0.005 - 0.00333T_{XI} \qquad (7.18)$$

T_{XI} is the average air temperature in Chabarovsk in October.

The maximum water level for an ice jam is determined by using the reduced discharge Q' (a hypothetical summer water discharge which raises water levels and would create an ice jam in winter) and summer curve $Q = f(H)$. In this case it is necessary to calculate the reductioned discharge Q' which can be estimated roughly as:

$$Q' = \frac{Q_{kr}}{k_p} \qquad (7.19)$$

Spring ice dams that build up on rivers break-up downstream under the influence of spring flood waves from the upper part of the basin. This phenomena is particularly important for south-north-flowing rivers in Canada and the northern parts of Europe and the Russian Federation. The ice break-up on these rivers takes the shape of a chain reaction of the consecutive formation and destruction of ice dams of varying magnitude.

The maximum ice dam water level on a given river reach depends on many factors, which can be divided into those related to the process of ice cover formation and those related to its destruction. The most powerful dams and catastrophic floods arise when high water levels occur during ice cover formation. This can arise when high flows in autumn meet a channel constricted by ice slush, especially if the freezing of the river is accompanied by some movement of the ice cover (Buzin and others, 1989). It can also occur when a rapid rise during a spring flood in the upper reaches coincides with a sharp cooling at the limit of the ice cover in the reach under consideration so that the ice becomes particularly durable and forms an ice dam in a downstream reach.

The factors of the first group make it possible in some river reaches, for example, on the Amur, Angara and Sukhona rivers, to predict maximum ice-dam water levels over periods of one to four months, using the following equation:

$$H_{t,dam} = 180 + 2.18\, H_x \qquad (7.20)$$

where $H_{t,dam}$ is maximum ice dam water level in centimetres; H_x is the water level at the period of ice cover formation in centimetres.

The addition of ice cover formation characteristics as parameters linked to the peculiarities of the spring processes has made it possible in the Lena river basin to predict the probability of occurrence of dangerous ice dam water levels during the ice break-up period for each of the four basic sections of the river with forecast lead times of 20 to 40 days. Using the relationship between ice thickness over the main part of the river and ice thickness at the main city within this part, it is possible to predict whether an ice dam will threaten the city or will form at another location. The probability of a correct prediction of dangerous levels is 80 per cent.

However, for many rivers where ice dams pose a particular risk, the development of methods for long-term forecasting is problematic and, where such forecasts are made, they frequently require correction. For this reason, a number of methods have been devised to provide short-term forecasts where ice dams occur each year. These forecasts are based on the physical and statistical relationships which take into account the major factors listed above. In a number of cases, a weather forecast for three to five days is taken into account to estimate the characteristics of ice durability and probability of cooling.

Some recommended methods relate to the forecasting of ice dam water levels for any part of the river, even in the absence of long observation series. In such cases, it is possible to use the discharge curve, $Q = f(H)$, and local meteorological data to calculate the relevant ice durability characteristics. The maximum water level at an ice dam is determined from $Q = f(H)$ and an appropriate Q' is calculated from equation 7.19. In this case, Q' is a conditional summer discharge, which could cause such a rise of water level, which occurs in ice dam formation. In equation 7.19, k_p is the winter factor for the period

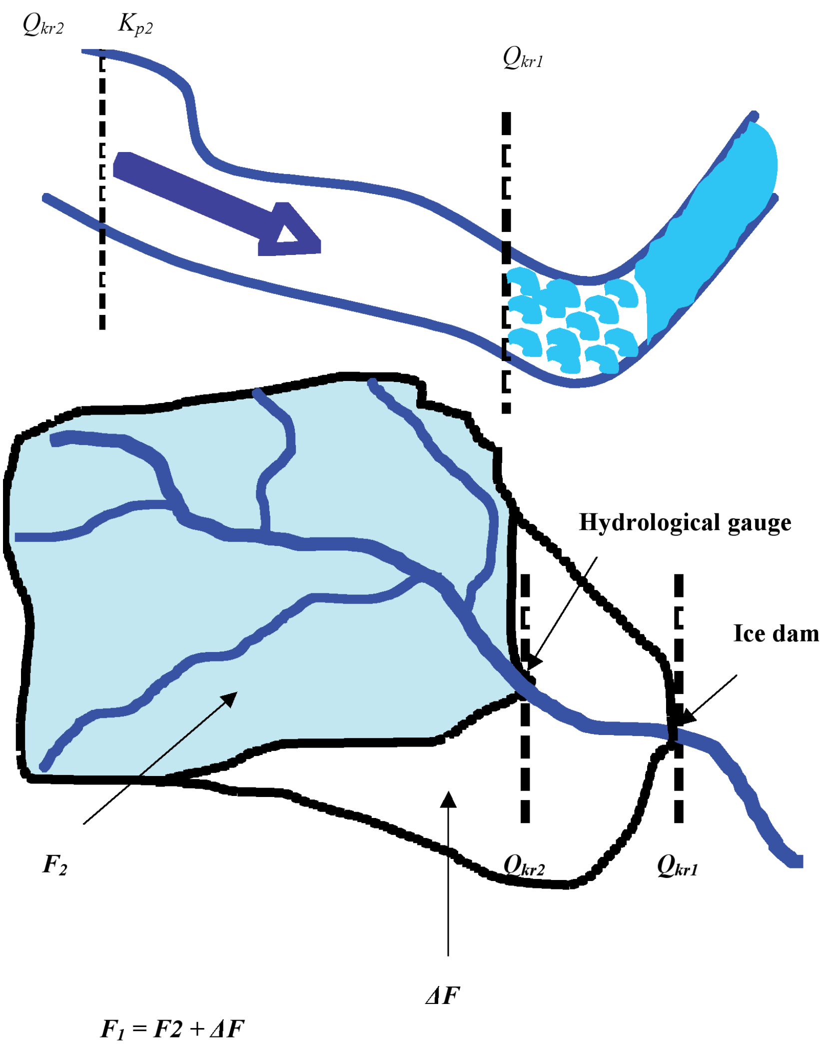


Figure II.7.9. River reach with an ice dam

of ice dam formation. This coefficient k_p is derived from the link with characteristics of an ice cover expressed by the following equation:

$$k_p = 8.13 \left(\frac{\varphi h_{ice}}{B}\right)^{0.38} (k_{ice} - 1) + 1 \tag{7.21}$$

where φ is the relationship between ice durability on the last day before ice cover break-up and ice durability on the day when snow disappears from the surface of the ice, which can be derived using techniques described in a number of publications (Buzin and others,1989); h_{ice} is the thickness of ice in metres before ice cover break-up; B is the width of the river in metres; k_{ice} is the winter factor at a maximum water level at the beginning of freezing in the autumn (for various river catchments, k_{ice} = 0.65 0.85). The winter factor can also be calculated as follows:

$$k_{ice} = 1 - \frac{1}{1.1 - 1 \log g F} \tag{7.22}$$

where F is the catchment area in km^2 located above an ice dam.

Where there is a river reach without tributaries above an ice dam, and there is a hydrological gauge in this part of the river (Figure II.7.9), it is possible to use the method of equivalent on ice phases discharges for the forecast of discharge at the ice dam (Q_{kr1}):

$$Q_{kr1} = k_{p2}\, Q_{kr2}\, F_1/F_2 \tag{7.23}$$

where k_{p2} is the winter factor for the date of ice dam formation in the upper section, Q_{kr2} is the discharge in the upper section, appropriated for a maximum ice dam level in the upper section according to summer curve discharges $Q = f(H)$, and F_1 and F_2 are the areas of the basin closed by the lower and upper sections of the reach, $F_1 = F_2 + \Delta F$.

The following Websites provide valuable information for hydrological forecasting services:

http://edc.usgs.gov/
http://k12science.ati.stevens-tech.edu/curriculum/drainproj/reference.html
http://nsidc.org/snow/
http://snr.unl.edu/niwr/
http://ulysses.atmos.colostate.edu/~odie/snowtxt.html
http://water.usgs.gov/
http://water.usgs.gov/listurl.html
http://www.afws.net/
http://www.cpc.ncep.noaa.gov/products/expert_assessment/threats.shtml
http://www.cpc.ncep.noaa.gov/products/fews/
http://www.dartmouth.edu/artsci/geog/floods/
http://www.epa.gov/ebtpages/water.html
http://www.hpc.ncep.noaa.gov/nationalfloodoutlook/
http://www.ibwc.state.gov/wad/rtdata.htm
http://www.iwr.usace.army.mil/
http://www.msc.ec.gc.ca/crysys/
http://www.ncdc.noaa.gov/ol/climate/climateextremes.html
http://www.nohrsc.nws.gov/
http://www.nws.noaa.gov/oh/hads/
http://www.nws.noaa.gov/ohd/hdsc/
http://www.nwstc.noaa.gov/HYDRO/RFS/NWSRFS.html
http://www.pecad.fas.usda.gov/cropexplorer/global_reservoir/
http://www.sce.ait.ac.th/programs/courses/IWRM/Online_references.htm
http://www.worldclimate.com/
http://www.wri.org/watersheds/

References and further reading

Anderson, E. A., 1973: National Weather Service River Forecast System: Snow Accumulation and Ablation Model, Programs and Test Data. NOAA NWSHYDRO Technical Memorandum 17.

Appolov, B. A., G.P. Kalinin and V.D. Komarov, 1974: Course on Hydrological Forecasting. Leningrad, Gidrometeoizdat, 418 pp.

Barrett, C.B., 1999: Successful Development and Engineering of Global Integrated Water Management Systems, American Society of Civil Engineers, International Activities Committee, Roundtable Discussion paper, Washington, DC, 1999.

———, 2003: WMO-NOAA Hydrologic Forecasting Course, 14 October–7 November 2003, Kansas City, Kansas.

Barrett, C.B. and others, 1985: Hydrology Subcommittee of the Interagency Advisory Committee on Water Data, Guidelines on Community Local Flood Warning and Response Systems, Springfield, 1985.

Bellerby, T., M. Todd, D. Kniveton and C. Kidd, 2001: Rainfall estimation from a combination of TRMM precipitation radar and GOES multispectral satellite imagery through the use of an artificial neural network. *Journal of Applied Meteorology*, 39(12):2115–2128.

Bergstroem, S., 1976: Development and Application of a Conceptual Runoff Model for Scandinavian Catchments. SMHI Rapporter No. RH07, *Hydrological Oceanography*.

Beven, K., 2002. Towards an alternative blueprint for a physically based digitally simulated hydrologic response modelling system. *Hydrological Processes*, 16:189–206.

Borovikova, L.N., Y.M. Denisov, E.B. Trofimova and I.D. Shencis, 1972: Matematicheskoe modelirovanie processa stoka gornyh rek (Mathematical modelling of the runoff formation for mountain rivers). Trudy Sredaznigmi, Vyp. 61(76).

Borsch S.V. and T.P. Silantjeva, 1987: Method of the short-term and medium-term forecast of opening of the rivers on the basis of the generalized dependence. *The methodical instructions*. Moscow, Hydrometeorological center of USSR, pp. 28.

———, 1989: The improvement of the method of account of an ice cover destruction at the rivers and reservoirs. Report of the Hydrometeorological Center of USSR. v. 309, pp. 113–120. (in Russian).

Box, G.E.P. and G.M. Jenkins, 1976: Time-series analysis: Forecasting and Control. San Francisco, Holden-Day.

Braga, B., Jr. and O. Massambani, (eds.) 1997: Weather Radar Technology for Water Resources Management. Montevideo, UNESCO Press, 516 pp.

Brier, G.W. and R.A. Allen, 1951: Verification of Weather Forecasts. Compendium of Meteorology, (T.F. Malone, ed.) *American Meteorological Society*, pp. 841–848.

Brun, E., P. David, M. Sudak and G. Brunot, 1992; A numerical Model to Simulate snow-cover stratigraphy for operational avalanche forecasting, *Journal of Glaciology*, 38(128):13–22.

Buzin V.A., G.I. Bolotnikov and A.M. Filippov, 1989: Ice jams on the rivers – methods of study, account and forecast. In book: Problems of nowadays of hydrology. Leningrad, Hydrometeoizdat, pp. 220–231 (in Russian).

Carpenter, T.M. and K.P. Georgakakos, 2004: Impacts of parametric and radar rainfall uncertainty on the ensemble streamflow simulations of a distributed hydrologic model. *Journal of Hydrology* (in press).

Carpenter, T.M., K.P. Georgakakos and J.A. Sperfslage, 2001: On the parametric and NEXRAD-radar sensitivities of a distributed hydrologic model suitable for operational use. *Journal of Hydrology*, 254:169–193.

Carpenter, T.M., J.A. Sperfslage, K.P. Georgakakos, T. Sweeney and D.L. Fread, 1999: National threshold runoff estimation utilizing GIS in support of operational flash flood warning systems. *Journal of Hydrology*, 224:21–44.

Carroll, T. R.,1995: Remote-sensing of snow in the cold regions. 1995. First Moderate Resolution Imaging Spectroradiometre (MODIS) Snow and Ice Workshop; NASA/Goddard Space Flight Center, Greenbelt, Maryland. 3–14.

Chagnon, S.A., 1999: Record flood-producing rainstorms of 17–18 July 1996 in the Chicago metropolitan area. Part III, Impacts and responses to the flash flooding. *Journal of Applied Meteorology*, 38:273–280.

Cluckie, I.D. and C.G. Collier, eds, (1991): Hydrological Applications of Weather Radar, Environmental Management, Science and Technology Series, Chichester, Ellis Horwood, 644 pp.

Cluckie, I.D. and D. Han, 2000: Fluvial Flood Forecasting, *Journal of Water and Environmental Management*. 14(8) Chartered Institution of Water and Environmental Management (CIWEM).

Colbeck, S.C. (ed), 1980: *Dynamics of Snow and Ice Masses*. New York, Academic Press, pp. 305–395.

Collier, C.G. and R. Krzysztofowicz, 2000: Special Issue on Quantitative Precipitation Forecasting. *Journal of Hydrology*, 239:1–337.

Collinge, V.K. and C. Kirby (eds), 1987: *Weather Radar and Flood Forecasting*. New York, Wiley, 296 pp.

Dabberdt, W.F., J. Hales, S. Zubrick, A. Crook, W. Krajewski, J. Christopher Doran, C. Mueller, C. King, R.N. Keener, R. Bornstein, D. Rodenhuis, P. Kocin, M.A. Rossetti, F. Sharrocks and E.M. Stanley Sr, 2000: Forecast issues in the urban zone, Report of the 10th Prospectus Development team of the U.S. Weather Research Program. *Bulletin of the American Meteorological Society*, 81(9):2047–2064.

Donald J.R., E.D. Soulis, N. Kouwen and A. Pietroniro, 1995: A land cover-based snow cover representation for distributed hydrologic models. *Water Resources Research*, 31:(4):995–1009.

Draper, N.R. and H. Smith, 1966: *Applied Regression Analysis*. New York, Wiley.

Evensen, G., 1994: Sequential data assimilation with a non-linear quasi-geostrophic model using Monte Carlo methods to forecast error statistics, *Journal of Geophysical Research*, 90(C5):10143–10162.

Finnerty, B.D., M.B. Smith, D.-J. Seo, V. Koren and M.G. Moglen, 1997. Space-time sensitivity of the Sacramento model to radar-gage precipitation inputs, *Journal of Hydrology*, 203:21–38.

Fleming, G., 1975: *Computer Simulation Techniques in Hydrology*. New York, Elsevier.

Fulton, R.A., J.P. Breidenbach, D.-J. Seo and D.A. Miller, 1998: The WSR-88D rainfall algorithm. *Weather and Forecasting*, 13(2):377–395.

Georgakakos, K.P., 2002: US corporate technology transfer in hydrometeorology. *Journal of Hydroinformatics*, 4(1):3–13.

Georgakakos, K.P., 2004: Analytical results for operational flash flood guidance. *Journal of Hydrology* (in review).

Georgakakos, K.P. and J.A. Sperfslage, 2004: Operational rainfall and flow forecasting for the Panama Canal watershed. Chapter 16 in *Rio Chagres, A Multidisciplinary Profile of a Tropical Watershed* (R.S. Harmon (ed.)). Dordrecht, Kluwer Academic Publishers, 16.1–16.13 (in press).

Georgakakos, K.P. and W.F. Krajewski (eds.), 2000: Hydrologic Applications of Weather Radar. Special Issue of *Journal of Geophysical Research – Atmospheres*, 105(D2):2213–2313.

Ginzburg B.M., S.V. Borsch and N.D. Efremova, 2001: Methods of the background long-range and medium-range forecasting of dates of opening of the rivers of the European territory of Russia. *Journal of Meteorology and Hydrology*, 11:67–78.

Ginzburg B.M., S.V. Borsch, N.D. Efremova and M.I. Silnitskaya, 1998: A method for backgraund long-term prediction of ice appearance dates for rivers and reservoirs in European Russia and Byelorussia. *Journal of Meteorology and Hydrology*, 6:84–91.

Ginzburg B.M., S.V. Borsch, N.D. Efremova, M.I. Silnitskaya and K.N. Polyakova, 2002: Methods of long-range and medium-range forecasting of dates of the spring flood maximal level on rivers on European territory of Russia. *Journal of Meteorology and Hydrology*, 11:81–92.

Ginzburg B.M. and M.I. Silnitskaya, 2000: Influence of the oceans surface of temperature distribution on the on terms of opening of the rivers of the European territory of Russia. *Journal of Meteorology and Hydrology*, 6:86–91.

Gray, D.M. and D.H. Male (Eds.), 1981: Handbook of Snow: *Principles, Processes, Management and Use*. Toronto, Pergamon Press.

Hedstrom, N.R. and J.W. Pomeroy, 1998: Measurements and modelling of snow interception in the boreal forest. *Hydrological Processes*, 12(10–11):1611–1625.4.

Hydrometeoizdat, 1989: *Guide on the hydrological forecasts*. The ice forecasts. Vol. 3. Leningrad, 168 (in Russian).

Jelesnianski, C.P., 1974: SPLASH II: General Track and Variant Storm Conditions. NOAA Technical Memorandum, NWS, TDL-S2. US Department of Commerce, March.

Joliffe, T. Ian and B.D. Stephenson, 2003: F*orecast Verification A Practitioner's Guide in the Atmospheric Sciences*. Wiley, 240 pp.

Jones, H.G., J. Pomeroy, D.A. Walker and R.W. Hoham, 2001: *Snow Ecology: an interdisciplinary examination of snow-covered ecosystems*. Cambridge University Press, 377 pp.

Koren, V.I., B.D. Finnerty, J.C. Schaake, M.B. Smith, D.-J. Seo and Q.Y. Duan, 1999: Scale dependencies of hydrology models to spatial variability of precipitation. *Journal of Hydrology*, 217:285–302.

Kottegoda, N.T., 1980: *Stochastic Water Resources Technology*, London, Macmillan Press.

Kovar, K., and H.P. Nachtnebel (eds), 1996: Application of Geographic Information Systems in Hydrology and Water Resources Management. IAHS Publication No. 235. Wallingford, IAHS Press, Institute of Hydrology, 711 pp.

Krzysztofowicz, R., 1992: Bayesian correlation score: a utilitarian measure of forecast skill. *Monthly Weather Review*,120:208–219.

Kummerow, C., W. Barnes, T. Kozu, J. Shiue and J. Simpson, 1998: The Tropical Rainfall Measuring Mission (TRMM) sensor package. *Journal of Atmospheric and Oceanic Technology*, 15(3):809–817.

Kuo, C.Y. (ed.), 1993: Engineering Hydrology. *American Society of Civil Engineers*, New York, pp. 989–994.

Mogil, H.M., J.C. Monro and H.S. Groper, 1978: NWS flash flood warning and disaster preparedness programs. *Bulletin of the American Meteorological Society*, 59:690–699.

Munich Re, 2001: Topics, Annual Review: Natural Catastrophes 2000, Munich.

Murphy, A.H. and R.L. Winkler, 1987: A general framework for forecast verification. *Monthly Weather Review*, 115:1330–1338.

Murphy, A.H., 1988: Skill scores based on the mean square error and their relationships to the correlation coefficient. *Monthly Weather Review*, 116:2417–2424.

Murphy, A.H., 1996: General decompositions of MSE-based skill scores: measures of some basic aspects of forecast quality. *Weather and Forecasting*, 124:2353–2369.

National Oceanic and Atmospheric Administration and the United Nations Department of Economic and Social Affairs, 2004: *Guidelines for Reducing Flood Losses*, 87 pp.

National Research Council (NRC), 1991: *Opportunities in the Hydrologic Sciences*. Washington, DC, National Academy Press, 348 pp.

Ogden, F.L., J. Garbrecht, P.A. DeBarry and L.E. Johnson, 2001: GIS and distributed watershed models, II, Modules, interfaces and models. *Journal of Hydrologic Engineering*, 6(6):515–523.

Overland, J.E., 1975: Estimation of Hurricane Storm Surge in Apalachicola Bay, Florida. NOAA Technical Memorandum, NWS-17, US Department of Commerce, March.

Palmer, W.C., 1965: Meteorological Drought, Research Paper No. 45, US Weather Bureau. Washington, DC.

Pielke, R.A., Jr and M.W. Downton, 2000: Precipitation and damaging floods, Trends in the United States, 1932-1997. *Journal of Climate*, 13:3625–3637.

Pomeroy, J.W. and D.M. Gray (eds.), 1995: *Snowcover Accumulation, Relocation and Management*. National Hydrology Research Centre, Saskatoon, Sask. Canada. NHRI Science Report No. 7, 144 pp.

Pomeroy, J.W. and E. Brun, 2001: Physical Properties of Snow. In: *Snow Ecology: An Interdisciplinary Examination of Snow-covered Ecosystems*. (H.G. Jones, J.W. Pomeroy, D.A. Walker, R.W. Hoham, eds). Cambridge, Cambridge University Press, pp. 45–126.

Rango, A., J. Martinec, J. Foster and D. Marks, 1983: Resolution in operational remote-sensing of snow cover. Hydrological Applications of Remote-sensing and Remote Data Transmission, Proceedings of the

Hamburg Symposium, August 1983. IAHS Publ. No. 145.
Riccardi, G.A., E.D. Zimmermann and R.-A. Navarro, 1997: Zonification of areas with inundation risk by means of mathematical modelling in the Rosario region Argentina. In: *Destructive Water, Water-Caused Natural Disasters, their Abatement and Control* (G.H. Leavesley, H.F. Lins, F. Nobilis, R.S. Parker, V.R, Schneider, F.H.M. van de Ven, eds). IAHS Press, Institute of Hydrology, Wallingford, 123–135.
Seo, D.-J. and J.P. Breidenbach, 2002: Real-time correction of spatially nonuniform bias in radar rainfall data using rain gauge measurements. *Journal of Hydrometeorology*, 3(2):93–111.
Shi, J.C., J. Dozier and H. Rott, 1994: Snow Mapping in Alpine Regions with Synthetic Aperture Radar. *IEEE Transcations on Geoscience and Remote-sensing*, 31(1):152–158.
Sittner, W.T. and K.M. Krouse, 1979: Improvement of Hydrologic Simulation by Utilizing Observed Discharge as an Indirect Input (Computed Hydrograph Adjustment Technique – CHAT). NOAA Technical Memorandum NWS HYDRO-38, February, US Department of Commerce.
Sittner, W.T., C.F. Schauss and J.C. Monro, 1969: Continuous hydrograph synthesis with an API-type hydrological model. *Water Resources Research*, 5(5).
Smith, M.B., K.P. Georgakakos and X. Liang, (eds), 2004*a*: The Distributed Model Intercomparison Project (DMIP). Special Issue of *Journal of Hydrology* (in press).
Smith, M.B., D.-J. Seo, V.I. Koren and co-authors, 2004*b*: The Distributed Model Intercomparison Project (DMIP): An Overview. *Journal of Hydrology* (in press).
Sweeney, T.L., 1992: Modernized areal flash flood guidance. NOAA Technical Report NWS HYDRO 44, Hydrology Laboratory, National Weather Service, NOAA, Silver Spring, MD, October, 21 pp. and an appendix.
Tachikawa, Y., B.E. Vieux, K.P., Georgakakos and E. Nakakita (eds), 2003: *Weather Radar Information and Distributed Hydrological Modelling*. IAHS Publications No. 282. Wallingford, IAHS Press, 323 pp.
Todini, E. and J.R. Wallis, 1978: A Real-time Rainfall Runoff Model for an On-line Flood Warning System. AGU Chapman conference on applications of kalman filtering theory and techniques to hydrology, hydraulics and water resources, Pittsburgh, Pennsylvania, 22–24 May 1978.
Twedt, T.M., J.C. Schaake and E.L. Peck, 1977: National Weather Service extended streamflow prediction. *Proceedings of the western snow conference*, Albuquerque, New Mexico, 19–21 April 1977.
US Bureau of Reclamation, 1960: Design of Small Dams. Appendix A, Washington, DC, pp. 413–431.
US Department of Commerce, 2002: Final Report on Hurricane Reconstruction Program, Central American and the Dominican Republic, May 2002.
United Nations Educational, Scientific and Cultural Organization (UNESCO), 1991: International Workshop on Storm Surges, River Flow and Combined Effects. 8–12 April 1991, Hamburg, A contribution to the UNESCO-IHP project H-2-2.
Urbonas, B.R. and L.A. Roesner, 1993: Hydrologic design for urban drainage and flood control. In: *Handbook of Hydrology* (D.R. Maidment, ed.), New York, McGraw-Hill Inc., 28.1–28.52.
Weaver, J.F., E. Gruntfest and G.M. Levy, 2000: Two floods in Fort Collins, Colorado, Learning from a natural disaster. *Bulletin of the American Meteorological Society*, 81(10):2359–2366.
World Bank, 2002: *2002 World Development Indicators*. Washington, DC, World Bank, 405 pp.
World Meteorological Organization, 1969: *Hydrological Forecasting*, Technical Note No. 92, WMO-No. 228, Geneva.
———, 1975: *Intercomparison of Conceptual Models Used in Operational Hydrological Forecasting*. Operational Hydrology Report No. 7, WMO-No. 429, Geneva.
———, 1982: *Long-range Water-supply Forecasting*, edited by M. Dyhr-Nielsen. Operational Hydrology Report No. 20, WMO-No. 587, Geneva.
———, 1986: *Intercomparison of Models of Snowmelt Runoff*, WMO-No. 646, Geneva, Switzerland.
———, 1990: *Hydrological Models for Water Resources System Design and Operation*. Operational Hydrology Report No. 34, WMO-No. 740, Geneva.
———, 1992: *Simulated Real-time Intercomparison of Hydrological Models*. Operational Hydrology Report No. 38, WMO-No. 779, Geneva.
———, 1994: *Guide to Hydrologic Practices*, Fifth Edition, WMO-No. 168. Geneva, pp. 721–724.
——— and the United Nations Educational, Scientific and Cultural Organization, 1997: *The World's Water, Is there enough?*.
Young, P.C., 1993: *Concise Encyclopaedia of Environmental Systems*, Pergamon Press, pp. 22.

www.wmo.int