Methods of hydrological computations for water projects

A contribution to the International Hydrological Programme

Report prepared by the Working Group — Project 3.1

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Although the total amount of water on earth is generally assumed to have remained virtually constant, the rapid growth of population, together with the extension of irrigated agriculture and industrial development, is stressing the quantity and quality aspects of the natural system. Because of the increasing problems, man has begun to realize that he can no longer follow a "use and discard" philosophy — either with water resources or any other natural resource. As a result, the need for a consistent policy of rational management of water resources has become evident.

Rational water management, however, should be founded upon a thorough understanding of water availability and movement. Thus, as a contribution to the solution of the world's water problems, Unesco, in 1965, began the first world-wide programme of studies of the hydrological cycle — the International Hydrological Decade (IHD). The research programme was complemented by a major effort in the field of hydrological education and training. The activities undertaken during the Decade proved to be of great interest and value to Member States. By the end of that period, a majority of Unesco's Member States had formed IHD National Committees to carry out relevant national activities and to participate in regional and international cooperation within the IHD programme. The knowledge of the world's water resources had substantially improved. Hydrology became widely recognized as an independent professional option and facilities for the training of hydrologists had been developed.

Conscious of the need to expand upon the efforts initiated during the International Hydrological Decade and, following the recommendations of Member States, Unesco, in 1975, launched a new long-term intergovernmental programme, the International Hydrological Programme (IHP), to follow the Decade.

Although the IHP is basically a scientific and educational programme, Unesco has been aware from the beginning of a need to direct its activities toward the practical solutions of the world's very real water resources problems. Accordingly, and in line with the recommendations of the 1977 United Nations Water Conference, the objectives of the International Hydrological Programme have been gradually expanded in order to cover not only hydrological processes considered in interrelationship with the environment and human activities, but also the scientific aspects of multipurpose utilization and conservation of water resources to meet the needs of economic and social development. Thus, while maintaining IHP's scientific concept, the objectives have shifted perceptibly towards a multidisciplinary approach to the assessment, planning, and rational management of water resources.

As part of Unesco's contribution to the objectives of the IHP, two publication series are issued: "Studies and Reports in Hydrology" and "Technical Papers in Hydrology". In addition to these publications, and in order to expedite exchange of information in the areas in which it is most needed, works of a preliminary nature are issued in the form of Technical Documents.

The purpose of the continuing series "Studies and Reports in Hydrology" to which this volume belongs, is to present data collected and the main results of hydrological studies, as well as to provide information on hydrological research techniques. The proceedings of symposia are also sometimes included. It is hoped that these volumes will furnish material of both practical and theoretical interest to water resources scientists and also to those involved in water resources assessments and the planning for rational water resources management.
### Contents

1 Introduction

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>11</td>
</tr>
<tr>
<td>1.2</td>
<td>12</td>
</tr>
<tr>
<td>1.3</td>
<td>13</td>
</tr>
<tr>
<td>1.4</td>
<td>14</td>
</tr>
<tr>
<td>1.5</td>
<td>15</td>
</tr>
</tbody>
</table>

**Tables**

| 1.1     | ENGLISH LANGUAGE REFERENCES REFERRED TO FREQUENTLY IN GUIDEBOOK 16 |
| 1.2     | SOURCES FOR ORDERING PUBLICATIONS REFERENCED BY THIS GUIDEBOOK 17 |
| 1.3     | PROCEEDINGS OF RECENT INTERNATIONAL HYDROLOGICAL CONFERENCES AND SYMPOSIA 26 |

REFERENCES 28

2 Overview of hydrological computations for water projects

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>31</td>
</tr>
<tr>
<td>2.1.1</td>
<td>Introduction 31</td>
</tr>
<tr>
<td>2.1.2</td>
<td>Meteorological Data 31</td>
</tr>
<tr>
<td>2.1.3</td>
<td>Hydrological Data 31</td>
</tr>
<tr>
<td>2.1.3.1</td>
<td>Stage Data 32</td>
</tr>
<tr>
<td>2.1.3.2</td>
<td>Discharge Data 32</td>
</tr>
<tr>
<td>2.1.3.3</td>
<td>Sediment Discharge Data 32</td>
</tr>
<tr>
<td>2.1.4</td>
<td>Data Time Intervals 32</td>
</tr>
<tr>
<td>2.1.5</td>
<td>Statistical Properties of Data 32</td>
</tr>
<tr>
<td>2.1.6</td>
<td>Data Generation 33</td>
</tr>
<tr>
<td>2.1.7</td>
<td>Data Quality Control 33</td>
</tr>
<tr>
<td>2.2</td>
<td>INTRODUCTION TO HYDROLOGICAL TECHNIQUES AND THEIR RELATION TO SPECIFIC TYPES OF WATER PROJECTS 34</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Purpose 34</td>
</tr>
<tr>
<td>2.2.2</td>
<td>Regional Analysis for Generation of Missing Data 34</td>
</tr>
<tr>
<td>2.2.3</td>
<td>Reestablishment of Natural Flows 35</td>
</tr>
<tr>
<td>2.2.4</td>
<td>Analysis of Flow Variability 35</td>
</tr>
<tr>
<td>2.2.5</td>
<td>Hydrological Modelling 35</td>
</tr>
<tr>
<td>2.2.6</td>
<td>Other Hydrologic Techniques 36</td>
</tr>
<tr>
<td>2.2.7</td>
<td>Effects of Water Projects 37</td>
</tr>
</tbody>
</table>
3 Hydrologic estimates required for specific types of water projects

Section

3.1 INTRODUCTION
3.1.1 Purposes and Objectives of Water Projects 43
3.1.2 Basic Types of Water Projects 44

3.2 HYDROLOGIC ESTIMATES REQUIRED FOR RESERVOIR PROJECTS
3.2.1 General Steps for All Reservoirs 45
3.2.2 Complementary Steps for Reservoirs with Flood Control Storage 46
3.2.3 Complementary Steps for Reservoirs with Hydroelectric Power Storage 46
3.2.4 Complementary Steps for Reservoirs with Conservation Storage 47
3.2.5 Complementary Steps for Multipurpose Reservoirs 47
3.2.6 Complementary Steps for Mixes of Reservoirs and Other Structural and Nonstructural Measures 47

3.3 HYDROLOGIC ESTIMATES REQUIRED FOR NONRESERVOIR PROJECTS
3.3.1 Flood Control Projects 48
3.3.2 Water Quality Control Projects 48
3.3.3 Irrigation Projects 49
3.3.4 Drainage Projects 49
3.3.5 Municipal and Industrial Water Supply Projects 50
3.3.6 Navigation Projects 51
3.3.7 Recreation and Fish and Wildlife Projects 51

3.4 HYDROLOGIC ESTIMATES REQUIRED FOR BASIN-WIDE LONG-TERM PLANNING FOR INTEGRATED DEVELOPMENT OF WATER RESOURCES 51

Table
3.1 Project Purposes, Their Objectives, and the Associated Structural Measures 54

References 55

4 Hydrological techniques available for water projects

Section

4.1 TECHNIQUES FOR REGIONAL ANALYSIS OF HYDROLOGICAL AND METEOROLOGICAL DATA 57
4.1.1 Introduction 57
4.1.2 Homogeneity of Data 57
4.1.3 Statistical Parameters 57
4.1.4 Hydrometeorological Mapping 58
4.1.5 Regression Analysis 58
4.1.6 Water Balance Ratios 59
4.1.7 Application to Inadequately Gaged Sites 59
References 60

4.2 DETERMINATION OF NATURAL FLOWS 62
4.2.1 Introduction 62
4.2.2 Data Inventory 62
4.2.3 Rainfall-Runoff Relations 62
4.2.4 Reservoir Routing 63
4.2.5 Channel Routing 63
4.2.6 Graphic and Regression Techniques 64
References 65

4.3 TECHNIQUES FOR EVALUATING ANNUAL FLOW VARIABILITY AND THE CHARACTERISTICS OF INTRA-ANNUAL FLOW DISTRIBUTION 67
4.3.1 Introduction 67
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.3.2 Flow Variability</td>
<td></td>
</tr>
<tr>
<td>4.3.2.1 Annual Flow Variability</td>
<td>67</td>
</tr>
<tr>
<td>4.3.2.2 Intra-Annual Flow Variability</td>
<td>67</td>
</tr>
<tr>
<td>4.3.3 Fitting Probability Distributions to Flow Data</td>
<td>67</td>
</tr>
<tr>
<td>4.3.3.1 Graphical Fitting</td>
<td>68</td>
</tr>
<tr>
<td>4.3.3.2 Method of Moments</td>
<td>68</td>
</tr>
<tr>
<td>4.3.3.3 Method of Maximum Likelihood</td>
<td>68</td>
</tr>
<tr>
<td>4.3.3.4 Other Methods</td>
<td>68</td>
</tr>
<tr>
<td>4.3.4 Application of Flow Data Distributions</td>
<td>68</td>
</tr>
<tr>
<td>4.3.4.1 Monthly Flow Distribution</td>
<td>69</td>
</tr>
<tr>
<td>4.3.4.2 Daily Flow Distribution</td>
<td>69</td>
</tr>
<tr>
<td>4.3.5 Absence of Observed Data</td>
<td>69</td>
</tr>
<tr>
<td>References</td>
<td>70</td>
</tr>
<tr>
<td>4.4 DETERMINATION OF EXTREME-DISCHARGE AND STAGE FREQUENCY CURVES FOR NATURAL AND REGULATED CONDITIONS</td>
<td>71</td>
</tr>
<tr>
<td>4.4.1 Introduction</td>
<td>71</td>
</tr>
<tr>
<td>4.4.2 Data Series</td>
<td>71</td>
</tr>
<tr>
<td>4.4.3 Homogeneous Events</td>
<td>71</td>
</tr>
<tr>
<td>4.4.4 Discharge-Frequency Curves</td>
<td>71</td>
</tr>
<tr>
<td>4.4.5 Peak-Discharge Frequency Curves for Natural Conditions</td>
<td>72</td>
</tr>
<tr>
<td>4.4.6 Peak-Discharge Frequency Curves for Regulated Conditions</td>
<td>72</td>
</tr>
<tr>
<td>4.4.7 Low-Flow Frequency Curves for Natural and Regulated Conditions</td>
<td>72</td>
</tr>
<tr>
<td>4.4.8 Determination of River-Stage and Reservoir-Level Frequency Curves</td>
<td>72</td>
</tr>
<tr>
<td>References</td>
<td>73</td>
</tr>
<tr>
<td>4.5 DESIGN FLOOD DETERMINATION</td>
<td>75</td>
</tr>
<tr>
<td>4.5.1 Introduction</td>
<td>75</td>
</tr>
<tr>
<td>4.5.2 Policy Considerations</td>
<td>75</td>
</tr>
<tr>
<td>4.5.3 Design Floods of Specified Frequency</td>
<td>75</td>
</tr>
<tr>
<td>4.5.4 Probable Maximum Flood</td>
<td>77</td>
</tr>
<tr>
<td>4.5.5 Conversion of Design Storm Precipitation to Runoff</td>
<td>78</td>
</tr>
<tr>
<td>4.5.5.1 Loss Rates</td>
<td>78</td>
</tr>
<tr>
<td>4.5.5.2 Snowmelt</td>
<td>78</td>
</tr>
<tr>
<td>4.5.5.3 Direct Runoff</td>
<td>78</td>
</tr>
<tr>
<td>4.5.5.4 Base Flow</td>
<td>79</td>
</tr>
<tr>
<td>4.5.5.5 Routing and Combining of Streamflows</td>
<td>79</td>
</tr>
<tr>
<td>4.5.5.6 Reservoir Routing</td>
<td>79</td>
</tr>
<tr>
<td>4.5.5.7 Digital Computer Applications</td>
<td>80</td>
</tr>
<tr>
<td>References</td>
<td>80</td>
</tr>
<tr>
<td>4.6 DETERMINATION OF FREEBOARD REQUIREMENTS FOR WIND AND WAVE ACTION</td>
<td>83</td>
</tr>
<tr>
<td>4.6.1 Introduction</td>
<td>83</td>
</tr>
<tr>
<td>4.6.2 Definition of Freeboard</td>
<td>83</td>
</tr>
<tr>
<td>4.6.3 Purpose</td>
<td>83</td>
</tr>
<tr>
<td>4.6.4 Channel and Levees</td>
<td>83</td>
</tr>
<tr>
<td>4.6.5 Large Lakes and Coastal Flood Barriers</td>
<td>84</td>
</tr>
<tr>
<td>4.6.6 Dam Embankments</td>
<td>84</td>
</tr>
<tr>
<td>4.6.7 Wind Induced Setup</td>
<td>84</td>
</tr>
<tr>
<td>4.6.8 Wave Height</td>
<td>84</td>
</tr>
<tr>
<td>4.6.9 Wave Run-Up</td>
<td>85</td>
</tr>
<tr>
<td>4.6.10 Design Water Level</td>
<td>85</td>
</tr>
<tr>
<td>4.6.11 Design Wind Characteristics</td>
<td>85</td>
</tr>
<tr>
<td>4.6.12 Contingencies</td>
<td>85</td>
</tr>
<tr>
<td>References</td>
<td>86</td>
</tr>
<tr>
<td>4.7 DETERMINATION OF WATER SURFACE PROFILES</td>
<td>87</td>
</tr>
<tr>
<td>4.7.1 Introduction</td>
<td>87</td>
</tr>
<tr>
<td>4.7.2 One-Dimensional Steady Flow</td>
<td>87</td>
</tr>
<tr>
<td>4.7.3 One-Dimensional Unsteady Flow</td>
<td>87</td>
</tr>
<tr>
<td>4.7.3.1 Hydrologic Techniques</td>
<td>87</td>
</tr>
<tr>
<td>4.7.3.2 Techniques Utilizing Portions of the Unsteady Flow Equations (St. Venant Eqs)</td>
<td>87</td>
</tr>
<tr>
<td>4.7.3.3 Techniques Utilizing the Full Unsteady Flow Equations</td>
<td>88</td>
</tr>
<tr>
<td>4.7.4 Two-Dimensional Steady and Unsteady Flow</td>
<td>88</td>
</tr>
<tr>
<td>References</td>
<td>89</td>
</tr>
</tbody>
</table>
4.8 HYDROLOGICAL BASIS FOR DETERMINATION OF RESERVOIR STORAGE REQUIREMENTS

4.8.1 Introduction 91
4.8.2 Flood Control Storage 91
4.8.3 Conservation Storage 92
4.8.4 Hydroelectric Power Storage 94
4.8.5 Sediment Reserve Storage 95
4.8.6 Inactive Storage 96
References 96

4.9 HYDROLOGIC TECHNIQUES FOR DETERMINATION OF RESERVOIR OPERATION PLANS 99

4.9.1 Introduction 99
4.9.2 General Analysis Procedure 99
4.9.3 Single Reservoir Analysis 100
4.9.4 Reservoir System Analysis 102
4.9.5 Simulation Models 103
4.9.6 Optimization Models 104
4.9.7 Reservoir Release Determination During Flood Emergencies 104
References 106

4.10 EFFECTS OF PROJECTS ON FLOW, EVAPORATION, SEDIMENT MOVEMENT, GROUNDWATER, AND ENVIRONMENTAL QUALITY 109

4.10.1 Introduction 109
4.10.2 The Effects of Projects on Flow 109
4.10.2.1 Reservoir Project 109
4.10.2.2 Nonreservoir Projects 109
4.10.2.3 Basinwide Systems 110
4.10.3 The Effects of Projects on Evaporation 110
4.10.4 The Effects of Projects on Environmental Quality 110
4.10.4.1 Reservoir Projects 110
4.10.4.2 Nonreservoir Projects 111
4.10.5 The Effects of Projects on Sediment Movement 111
4.10.5.1 Reservoir Projects 111
4.10.5.2 Nonreservoir Projects 112
4.10.6 The Effects of Projects on Groundwater 112
4.10.7 The Effects of Projects on Water Temperature 112
4.10.7.1 Reservoir Projects 112
4.10.7.2 Nonreservoir Projects 112
4.10.8 The Effects of Projects on Ice 113
4.10.8.1 Reservoir Projects 113
4.10.8.2 Nonreservoir Projects 113
4.10.9 Use of Computer Programs for Water Quantity and Quality Investigations 113
References 114

Appendix

Generalized Mathematical Models 117
1. HEC-1 117
2. SSARR 118
3. Hydrological Simulation Model 118
4. SWMM 118
5. Tank Model 119
6. Constrained Linear System 119
7. STORM 119
8. WQRRS 120
9. HEC-5 120
References 121

Tables

4.5-1 Design Criteria for Hydraulic Structures 75
4.5-2 Reservoir Flood and Wave Standards by Dam Category 76
1 Introduction

1.1 OBJECTIVES AND SCOPE OF THE GUIDEBOOK

The planning and design of water resources projects relies heavily upon hydrologic computations. The objective of this publication is to provide guidance on hydrological analyses and computations which are required for these purposes. The guidebook is prepared for engineers, hydrologists and other specialists, particularly those from developing countries, who seek advice on currently available methods. State-of-the-art hydrologic techniques are briefly described, and alternative techniques are presented along with the conditions under which each technique may or may not be useful. In addition, the guidebook cites widely available international and national publications containing information on currently used practical hydrologic techniques that can be used in developing the hydrologic estimates required for various types of water resource projects. References to a number of widely available computer programs which should be useful as practical tools in developing the hydrologic estimate are also presented.

This guidebook is not intended to present the detailed computations required in order to develop the hydrologic estimates, but rather to provide information on where details of alternative techniques can be found in widely available publications, and to indicate which alternative technique may provide a more realistic answer under certain prescribed conditions.

The hydrological computations discussed in this guidebook deal mainly with the use of surface water resources. Computations associated with the use of groundwater resources are not treated herein.

This guidebook has been prepared by the International Hydrological Program Working Group on Methods of Computation of Hydrological Parameters for Water Projects (IHP-Project 3.1) under the Chairmanship of Prof. A.A. Sokolov. The Working Group consisted of:

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Mr. G.A. Schultz (Federal Republic of Germany)
Mr. A.A. Sokolov (USSR)
Mr. R.M. Vallejos (Argentina)

The Working Group elected Mr. B.S. Eichert, Mr. J. Kindler, Mr. G.A. Schultz and Mr. A.A. Sokolov as editors of the guidebook. WMO contributed to the activities of the working group and participated actively in the editorial group through Mr. A.J. Askew of the WMO Secretariat and Mr. B. Wingard from Norway. The final editing and the preparation of the manuscript in camera-ready form were carried out under the supervision of Mr. B.S. Eichert.
1.2 STRUCTURE AND USE OF THE GUIDEBOOK

The guidebook is divided into four chapters and one appendix. Chapter 1 sets the background for the remainder of the guidebook by indicating the scope and intended readership, by relating hydrology to water resources management, and by describing the basic steps in water resources development (planning, design, construction and operations).

Chapter 2 on Overview of Hydrological Computations for Water Projects presents brief background information on hydrologic and meteorological data and its processing and briefly introduces the hydrologic techniques used in developing the hydrologic estimates necessary for various types of water resources projects. Table 2.1 relates the hydrological techniques briefly described in Chapter 2 (and critically analyzed and referenced in Chapter 4), with the grouping of types of projects used by Chapter 3 (where lists of all hydrologic estimates needed for each type of project are given).*

Chapter 3 on Hydrologic Estimates Required for Specific Types of Water Projects presents brief background information on various types of water resources projects and project purposes. In addition, Chapter 3 presents detailed lists of specific hydrologic estimates that are generally required for planning various types of projects. For each of the hydrologic estimates shown in Chapter 3, a cross-reference is given to the appropriate part of Chapter 4 where techniques for determining that estimate are critically reviewed and appropriate references are cited.

Chapter 4 on Hydrologic Techniques Available for Water Projects presents brief summaries and critiques of alternative techniques for determining the hydrologic estimates listed in Chapter 3 along with appropriate references to widely used international and national publications where the details of these techniques may be found. Chapter 4 also references some computer programs which are widely used in determining the hydrologic estimates required for various water resources projects.

The appendix to Chapter 4 briefly describes and provides references for selected generalized computer programs which should be useful for determining many of the hydrologic estimates.

The following procedure is recommended to make the most efficient use of the guidebook.

1) Review Chapters 1-3 and get generally acquainted with the introductory material and guidebook organization.

2) Check to see if you have access to the necessary major references to describe the hydrologic techniques you anticipate using (see Table 1.1). If not, order the relevant publications using Table 1.2 to locate the addresses.

3) Turn to Table 2.1 and locate the type of project for which hydrologic analyses are required (e.g., hydroelectric reservoir). Determine, from Table 2.1, which sections of Chapter 3 are needed for that type of project (e.g., Sec. 3.2.1 for all reservoirs and 3.2.3 for hydroelectric reservoirs). Using Table 2.1 determine which hydrologic techniques need to be used and where those techniques are discussed in Chapter 4. This determination provides an overview of the necessary work required (e.g., techniques will be needed which are discussed in Sec. 4.1-4.10).

4) Next, turn to the sections in Chapter 3 which are needed for your selected type of project (e.g., Sec. 3.2.1 and 3.2.3 based on Table 2.1 as determined above or from the table of contents for Chapter 3). Under the appropriate parts of Chapter 3, you will find a list of specific hydrologic estimates that need to be made to design the project (e.g., under 3.2.1, 15 estimates are listed and under 3.2.3, 7 estimates are listed). For each estimate required, determine the appropriate references (e.g., item 1 under 3.2.1 refers to regional studies contained in Sec. 4.1).

5) Now look at the Chapter 4 material that is referenced for each hydrologic estimate and determine the appropriate references that you will need to give you the details of the technique necessary to make the required estimate.

6) Determine the hydrologic estimate using the appropriate references from Chapter 4.

References in this guidebook are presented at the end of Chapters 1-3 and at the end of the Appendix to Chapter 4. However, Chapter 4 references are found at the end of each subchapter (i.e., 4.1, 4.2, etc.) because of the large number of references for each subchapter and since each subchapter concerns a major hydrologic computation which can be studied separately from those in the other subchapters.

*The use of the terms "hydrologic estimate" and "hydrologic technique" may cause the reader some confusion if he tries to distinguish between them too much. In some cases, lists of hydrologic techniques (such as Table 2.1) may also represent lists of hydrologic estimates (i.e., a spillway design flood is a hydrologic estimate that is needed and there are several hydrologic techniques for calculating the spillway design flood).
1.3 ASSOCIATION WITH OTHER PUBLICATIONS

In order to keep the volume of the present guidebook within reasonable limits, it was decided that this guidebook would not present detailed hydrologic techniques, but would rely on the many widely known international and national publications which provide this kind of information. The guidebook's main use then is to identify which hydrologic estimates are required for certain types of projects (Chapter 3) and to cite the specific publications where the detailed techniques are presented (Chapter 4 for hydrologic techniques and Chapter 3 for non-hydrologic techniques). The primary association of this guidebook with other publications is therefore based on material in Chapter 4. The key English-language references (English being the working language of the IHP Working Group) cited in Chapter 4 were used to construct Table 1.1 which reflects the most frequently cited references in the guidebook. In order to make the best use of this guidebook, it is therefore important (for the English reader) to at least have access to the references presented in Table 1.1. If the reader expects to concentrate on only a few subchapters, then he should try to obtain most of the references under those subchapters. Table 1.2 provides mailing addresses where these publications may be obtained. References to specific publishers in Table 1.2 are made on the right hand side of the reference list for each chapter or subchapter. In addition to the references used in the guidebook, numerous additional publications contain similar material, but were not included in the guidebook because these publications were:

1) Simply duplicative in nature to the ones shown,
2) Not known to the Working Group (particularly those that were not in English),
3) Not considered to be widely available to the international community.

Much of the material which is widely available to international audiences (references in Chapter 4 being the exceptions) is geared toward the more academic or scientific audiences than to the practicing engineer engaged in planning a water resources project.

Unesco, within the framework of IHP-II Project A.2.10 is now preparing a Casebook of examples on methods for computing hydrological parameters for water projects. This project continues the work of the Guidebook and will give examples of hydrological computations for the planning and design of various water projects. In order to be of practical and direct use to planners, designers and engineers, the Casebook will also include procedures and examples from actual water projects and highlight the specific hydrological computations for these projects.

During the IHD-IHP period, a wide exchange of ideas took place on the main problems of hydrology and on the scientific achievements in the field of hydrological computations. Numerous international scientific conferences, symposia, seminars, and sessions of working groups were held which were organized by Unesco, WMO, IAHS and by a great many of the IHP National Committees. Proceedings of many of the International conferences and symposia are contained in Table 1.3.

An international symposium on specific aspects of hydrological computations for water projects (on the very subject of the guide) was convened by Unesco in cooperation with WMO and IAHS and the USSR National Committee for the IHP in Leningrad (September, 1979). At this symposium, more than 70 scientific reports were presented from the IHP Member-States. The proceedings of this symposium are being published in the Unesco series "Studies and Reports in Hydrology" in the Soviet Union.

Results of hydrological research, and data obtained from hydrological measurements, have been summarized in a number of international publications. Noteworthy from the point of view of hydrological science and engineering practice are those on representative and experimental basins (Toebes, 1970), methods for water balance computations (Sokolov, 1974), data collection and processing for flood studies (Snyder, 1971), nuclear techniques in hydrology (International Atomic Energy Agency, 1968), hydrological maps (Unesco, 1977), water quality investigations (CMEA, 1977), and methods for flood flow computation (Sokolov, 1976).

A number of publications have been prepared by WMO on aspects of operational hydrology, such as "Meteorological and Hydrological Data Required in Planning the Development of Water Resources" (Andrejanov, 1975), "Guide to Hydrological Practices" (WMO, 1974), etc. The recent progress in hydrological knowledge is also reflected in a series of hydrological handbooks (Viessman, 1977; Dubreuil, 1974; Heras, 1972; and National Environmental Research Council, 1975).

Various organizations of the UN family have also made important contributions in the field of water quality monitoring, data processing and interpretation. These are reflected in the following publications: "Water Quality Surveys" (Unesco-WHO, 1978), "GEMS/Water Operational Guide" (UNEP, 1978), and others. Important contributions to hydrology have been made by the monograph, "World Water Balance and Water Resources of the Earth," and "Atlas of the World Water Balance" (Unesco, 1978b, Unesco, 1978a).
A "Manual on methods of computation of water balances of large lakes and reservoirs", Vol.1, Methodology; (Unesco, 1981), "Casebook on Low Flow Computation", and the technical paper "Investigation of water regime of river basins affected by irrigation," have been prepared for publication within the framework of the International Hydrological Programme.

A recent very relevant development in WMO's activities has been the initiation of the Hydrological Operational Multipurpose Subprogramme (HOMS). This provides a means for the transfer between Member countries of a whole range of technology relating to operational hydrology, including that required for the collection, transmission and processing of hydrological data for use in the design and operation of water projects. The technology is transferred as a series of components in the form of guidance manuals, instrument specifications and computer software. These components are available to all National Hydrological and Meteorological Services and, in a manner similar to that described in Chapter 3 of this Guidebook, they can be compiled into sequences which can be used to satisfy the various needs of different water projects.

1.4 HYDROLOGY AS A SOURCE OF INFORMATION FOR WATER RESOURCES MANAGEMENT

Hydrology is the basic science underlying water resources management since water cannot be managed without knowing the quantity, aerial distribution and timing of the movement of water. Hydrology has evolved significantly in recent years due in part to the improvements in data collection systems, i.e., from historical records of rare events (floods, droughts, ice regime anomalies, etc.) to systematic observations on ever-expanding network of stations. At present in the world, there are more than 60,000 stream-gaging stations, and over 100,000 meteorological stations where systematic observations are made on the elements of the hydrometeorological regime of rivers, lakes, reservoirs, oceans and ground water. In some countries, the procedures used in the design and operation of networks of hydrometeorological stations are prescribed by special instructions and guides (Gidrometeoizdat, 1960, 1969, 1975, 1978, 1973; Gidrometeoizdat, 1973; U.S. Geological Survey, 1978).

The development of hydrological data networks has greatly increased the knowledge of world water resources. Gaps in the world and national maps on the hydrologic elements are becoming smaller. Hydrological interrelations with meteorology, hydrogeology and other sciences are becoming better understood, which enables improved regional and global studies of all the elements of the hydrological cycle.

One of the main aspects in practical application of hydrology is the computation of basic elements of the hydrological regime, including characteristics of streamflow, precipitation, evaporation, dynamics of the water masses, sediment transport and discharge, water quality, etc., which are essential for designing, constructing and operating water projects and hydraulic structures. Safety, cost and efficiency of projects and structures greatly depend on the reliability of hydrological estimates.

Reliable determination of hydrological estimates is complicated because of the need to account for possible time-series variations (for example, variations of flows and sediments) for different structures with variable project lives. General principles for the solution of these problems for planning water projects are presented in a number of publications (Kritsky, 1952; Kritsky, 1977; Maass, 1962). Long-term hydrological and meteorological data series, published in hydrological and meteorological yearbooks, bulletins, etc., are of the utmost importance in making hydrological estimates. Many difficulties arise due to inadequate or missing data.

In addition to national and regional publications of flow data, many long-term observations from the world network of hydrological stations have been published since 1965 by Unesco in the series, "Discharges of Selected Rivers of the World," (Unesco, 1965).

Until recently, the solution of individual water problems was done on a local basis. Many contemporary water projects, however, are not isolated or intended for a single purpose (as, e.g., hydroelectric power reservoirs or connecting navigation canals). Projects may involve interrelated systems covering entire basins and regions. These water projects can infringe upon the interests of many branches of a national economy and have environmental, economic and social impacts over vast areas. Such large-scale projects often introduce significant changes in the environment. They may affect landscape and biota, change the water and salt balance of inland seas, and create potential for a change in climate and water circulation in the atmosphere.

The understanding and prediction of man-induced changes in water regime and water balance is a complicated problem of hydrology. Whereas some preliminary solutions regarding the effects of specific changes such as urbanization and deforestation on hydrology have been
attempted (Unesco, 1974 and 1980; FAO, 1969; Shiklomanov, 1975), the effects of complex interrelated changes are extremely difficult to predict. The complexity of this problem is due in part to the fact that a large-scale interference in nature may not be discovered immediately. Decades may pass before any negative and irreversible change in nature is revealed. Estimation and prediction of possible changes in the environment is an essential aspect of contemporary water resources management.

Measures to protect the environment against pollution and depletion require scientific substantiation. For example, the effects of urbanization, forest cutting and afforestation, plowing and agrotechnical measures, irrigation and drainage, mining, acid precipitation, and other types of man's activity in channels and river basins that cause changes in water regime and balances must be recognized and evaluated. A contribution to the solution of these complex hydrological problems may possibly be obtained from the rapid development of remote sensing applications to hydrology and water resources management (Reeves, 1975; Kudritsky, 1977; Kupryanov, 1976; Wiesnet, 1979).

Development of water resources is affected and regulated by existing water laws. Such water laws affect priorities for water resources development. For example, laws may require that regional and basin-wide schemes for multipurpose development of water resources be prepared for current and future demands for all the sectors of the national economy on the basis of forecasts of economic and social development (Environment Canada, 1975). The prediction of water regime and balance changes is an integral part of such schemes and is the basis for a scientific substantiation of measures that will provide the most effective water resources use and protection.

1.5 PRINCIPAL STAGES OF WATER RESOURCES DEVELOPMENT (PLANNING, DESIGN, CONSTRUCTION AND OPERATION)

The development of a water resources project is a complex task that requires a minimum of three to five years from conception to construction and normally takes several times longer for large projects (James, 1971). The development is normally divided into the stages of planning, design, construction and operation. The planning stage is the principal stage discussed in this guidebook since hydrologic computations are so fundamental in the planning process. While hydrological computations are used in the design and construction stages, they have less impact on these stages and are not discussed in depth here. Although hydrologic computations are of paramount importance in the operation of many types of water resources projects, they are discussed in this guidebook only to the extent that they affect the planning process.

The separation of the four stages of project development is fairly straightforward except for the separation between the planning and design stages. The design stage can be thought of as the translation of the plan (from the planning stage) into the specific drawings and reports required for construction. The operations phase, while separate from the planning phase, is still greatly dependent upon the criteria, assumptions, and plans developed under the planning phase. It is certainly too late, after the project has been constructed, to start thinking about how the project will be operated. In most cases, the operation of the project will have major effects on the type, size, costs, and benefits of the project.

The specific steps with which a project proceeds from conception to construction vary from country to country and among agencies within the same country. However, James and Lee, (1971) provides some steps which are generally applicable to most projects and can be summarized as follows:

1) Identify need for project,
2) Authorize study,
3) Hold public hearings,
4) Perform feasibility study to determine if project is justified,
5) Get local public assurances for financial and moral support for project,
6) Get review of project by concerned parties,
7) Get authorization for construction,
8) Get funds appropriated for design and construction,
9) Collect additional data and perform more detailed studies,
10) Prepare final plans specifications and cost estimates,
11) Select contractors and construct the project, and
12) Turn project over to operations and maintenance.
Table 1.1

English Language References Referred to Frequently In Guidebook

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(1) Out of print in 1980, (may be reprinted) library copy may be available.
(2) A fourth edition is to be published in two volumes in 1981.
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Table 1.2

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OR

International Association of Hydrological Sciences (IAHS)
Blackwell Scientific Publications, Ltd.
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Oxford OX2 OEL
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Publications Room A-3315
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CHAPTER 1

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(1) out of print, library may have copies.
(2) available in limited quantities.
(3) Volume 2 and Volume 3.1 are out of print, library may have copies.
(4) Out of print, library may have copies.
(5) Not available separately, cost is approximately $110 U.S. dollars.
(6) Order from USGS in Virginia, USA.
2 Overview of hydrological computations for water projects

2.1 HYDROLOGICAL AND METEOROLOGICAL DATA AND THEIR PROCESSING

2.1.1 Introduction

Hydrological and meteorological data obtained from networks form the main basis for hydrological computations for water projects. Thus, the quality of such computations depends on the availability of hydrological and meteorological data, their resolution in time and space as well as their accuracy (Klemes, 1973; and McMahon, 1979). Where these data are inadequate, any project activity should start with design and the implementation of a data network for the project (WMO, 1976).

Where hydrological yearbooks are available, they may be the first source of information. For most projects, however, it is necessary to use the basic data (data lists, charts, tapes, etc.). Retrieval and processing of these data may be more efficient when the data required are in computer compatible form (Roche, 1968). Occasionally, data may be available in a ready to use (compiled and processed) form. However, updating of the data is also required in such cases.

2.1.2 Meteorological Data

If a dense streamgaging network is available with long observed time series at high resolution, this information is often sufficient as a basis for many types of hydrological computations for water projects.

For certain problems, however, meteorological data are of great value (WMO, 1976; WMO, 1974; and WMO, 1975b), e.g., rainfall data for the computation of extreme floods by use of rainfall-runoff models, for filling in gaps in runoff data series, etc. Furthermore, data on evaporation or other meteorological data (radiation, humidity, wind) which are required for the computation of evaporation are often very valuable, e.g., for computation of reservoir storage requirements and irrigation losses (Ratkovich, 1976; and Syroezhin, 1974).

Measurements on snow and snowmelt may aid in forecasting future flows from snowmelt. Meteorological data such as rainfall, air temperature, solar radiation, wind speed, etc., are also used to simulate or estimate snow water content, snowmelt and ice conditions in rivers and lakes. Snow measurements (depth, water content and aerial distribution) will also aid in calculating future runoff from snowmelt. In many countries, ice on rivers, lakes and reservoirs may cause problems for navigation or structures. Measurements of ice cover, e.g., ice thickness and other ice phenomena, may aid in forecasting future conditions.

2.1.3 Hydrological Data

A number of types of hydrological data are used for water resources computations. The most frequent are stage, discharge and sediment data, which are dealt with in more detail below. Other data, such as water velocities, water temperature, chemical and physiographic data, are also important.

It is advisable to construct a data bank for both meteorological and hydrological data before computations begin on a major water resources project. This bank, which is a permanent file (sometimes a computer-based data management system), should contain both raw observed data, as well as data which have undergone quality control and the necessary data processing.
2.1.3.1 Stage Data

Stage data are collected with the aid of various types of river gages. Types of gages, as well as the numerous problems associated with measurement, are discussed in many textbooks such as WMO (1974), Chow (1964), and United States Geological Survey (1978) and will not be discussed here.

2.1.3.2 Discharge Data

The transformation of river stage observations into discharge values is done with the aid of the nonlinear stage-discharge relationship (rating curve), which is established by means of discharge measurements (e.g., with current meters). Since only a few discharge measurements are typically available for a gaging site and since measurements during floods are rare, the rating curves are often not very reliable, especially if they are extrapolated for higher values. Potential changes in the hydraulic conditions at the gage (e.g., due to sedimentation, erosion, vegetation, ice conditions) are also frequent sources of errors. Another problem lies in the fact that the rating curves for highly unsteady flow (e.g., floods) sometimes occur in the form of loops. Further information on all of these problems may be acquired from textbooks such as Chow (1964) and Roche (1963).

2.1.3.3 Sediment Discharge Data

Data for sediment discharge, i.e., bed load and suspended load are difficult to collect and are not very accurate. It is generally possible to collect bed load continuously with time. Furthermore, sediment measurements are costly and the need for such measurements should be well established. Collection of sediment data is described in the literature (WMO, 1974; Chow, 1964; and Davis, 1969). Determination of sediment accumulation in reservoirs is discussed in Sec. 4.8.5.

The analysis of mean annual data of suspended sediment load may be based on graphs of relationships between mean annual discharge and suspended sediment load (Chow, 1964 and Davis, 1969).

2.1.4 Data Time Intervals

In order to achieve the most accuracy from hydrological computations, it is generally desirable to have continuous data records (automatic recorder) available and to carry out the required computations on the basis of a continuous time series. In many cases, this is not necessary because:

1) the observed data exist only for specified points in time or for specific time intervals (e.g., daily, weekly, monthly, data);
2) computations with small time intervals (e.g., hourly) are sometimes too expensive even with high-speed computers;
3) larger time intervals (e.g., monthly values for computation of required storage capacity of a water supply reservoir) are often sufficient.

The selection of both measurement interval and computational interval influences the accuracy of hydrological estimates and must be considered separately and carefully.

2.1.5 Statistical Properties of Data

Hydrological and meteorological data can be considered as statistical samples taken from the long time series generated by hydrological and meteorological processes in nature which represent the statistical population (Dyck, 1976; Svanidze, 1977; Rozhdestvenski, 1974; Brunet-Moret, 1969; Yevjevich, 1972a; Yevjevich, 1972b; and Shen, 1976).

Important parameters for water projects are (among others):
- average annual flow;
- distribution of streamflow during a year (over seasons, periods, months, days);
- peak discharges and maximum discharges for various durations:
  a) caused by snowmelt (maxima of snowmelt floods);
  b) caused by rainfall (maxima of storm floods);
  c) caused by both (rainfall and snowmelt);
- volumes of runoff from snowmelt and rainfall floods;
- minimum discharges:
  a) minimum mean daily discharges for separate seasons such as summer and winter periods;
b) minimum mean monthly discharges for separate seasons;
c) duration of droughts and freeze-up periods;
- annual suspended sediment or bedload discharge;
- distribution of suspended sediment or bedload discharge during a year;
- duration of ice phenomena, i.e., ice cover, ice drift.

These parameters can be compiled from time series obtained by long-term observations. In order to apply standard statistical techniques, the data have to be checked for stationarity and homogeneity, the techniques for which are given in Sec. 4.1. If the data are stationary, standard techniques of probability theory and time series analysis (stochastic processes) can be applied. Nonstationarity or nonhomogeneity of statistical series may be due to:

a) combination of hydrological characteristics with different origin into one series (e.g., maximum rainstorm discharges with maximum snowmelt flood discharges);
b) effect of man's activity (IAHS, 1977; Rodier, 1974; and Shiklomanov, 1975) on the conditions of runoff formation (e.g., urbanization).

Methods for detection of nonstationarity and nonhomogeneity, as well as methods to overcome these problems are given in Sec. 4.1. Another problem is the potential inaccuracy of observed data due to factors such as: a change of gaging site or zero flow level at gage, inaccuracy of the measuring devices or of measurements, or a change in velocity distribution due to vegetation or ice cover. Furthermore, the use of short observation periods may cause inaccuracies (time sampling errors) since the observation period may not be typical for the population characteristics.

Often the opinion is expressed that, if data are inadequate (low in quality or quantity), simple methods may be applied. Quite often the contrary is true, because it is necessary to apply relatively sophisticated methods if data are inadequate, in order to extract as much information from the data as possible.

A useful summary on "meteorological and hydrological data required in planning the development of water resources" is given in WMO Report No. 419 (WMO, 1975b). The report contains a glossary on hydrometeorological characteristics and parameters for design of water resources systems that may be especially valuable for planning purposes.

2.1.6 Data Generation

It may be useful to apply data generation techniques to synthesize or generate hydrological data in cases where:
1) there are gaps in the series of observed data,
2) the observation period is short,
3) data are not available at the site of interest (e.g., dam site) but in the neighboring region.

These techniques can:
1) fill in gaps by using information from:
   a) neighboring stations, e.g., by cross-correlation or regression techniques (Chow, 1964; Dyck, 1976; Svanidze, 1977; Fiering, 1971; and Fleming, 1975)
   b) rainfall stations.
2) extend short time series by cross-correlation, or regression with:
   a) other river gage data
   b) rainfall data
3) generate other data time series which are approximately equivalent to the observed series (in a statistical sense). Such generated records represent alternative sequences of possible future flows that are about as likely to occur as the historical flows. These techniques are discussed in Sec. 4.1, 4.8 and 4.9.

2.1.7 Data Quality Control

The foundation for any study using hydrological or meteorological data is good data. Before data are used in any study, they should be reviewed from a qualitative point of view. Methods of avoiding, diminishing, detecting and correcting errors in the data are called data quality control (WMO, 1974). Data errors may be assigned to two general categories, accidental and systematic.

Accidental errors are the result of errors by the observer, misinterpretation of the observer's notes, and mistakes in processing or transcribing the data. Errors can also result from physical phenomena such as the effect of atmospheric pressure on the water levels
in wells or the effect of a seiche on lake level readings. Such errors tend to be distributed around the mean value; therefore, these errors tend to cancel out when a large number of observations are used to obtain the average value.

Accidental errors can be minimized by careful verification of the data to eliminate transcribing errors. Large anomalies should be reviewed to ascertain whether they are the result of errors. The apparent anomaly may be the result of having a small sample and a comparison should be made with other information in the region. There are differing opinions on how to treat the so-called "outliers."

Systematic errors can result from a bias by the observer, a bad instrument, the exposure of the gage and data analysis procedures. These errors are not easily detected by the data analyst. The analyst can be aware of gage locations and the systematic bias that can result from changing land use patterns, upstream water developments, altered river courses, etc. Depending on the use of the data and the availability of manpower and funds, any known systematic errors should be corrected to provide a consistent data set.

Data quality control methods as defined above are infrequently described in references. Error detection and correction methods are only treated in a few instances (Herschy, 1978), and then only briefly. Unfortunately, no procedures comprising several sites or methods are covered as a whole, but are separated in the hydrologic literature. It seems necessary, therefore, to consult the national hydrologic data processing institutes to gain information on applied methods for data quality control.

2.2 INTRODUCTION TO HYDROLOGICAL TECHNIQUES AND THEIR RELATION TO SPECIFIC TYPES OF WATER PROJECTS

2.2.1 Purpose

The purpose of this section is to briefly introduce the main hydrological techniques, which are discussed in more depth in Chapter 4, and relate them through Table 2.1 to the various types of water resources projects as discussed in Chapter 3. The classification of various types of water resources projects, used in Chapter 3, are listed as "PROJECTS" in Table 2.1. The hydrological techniques, presented in Chapter 4, are listed under "HYDROLOGIC TECHNIQUES" in the first column of Table 2.1. The x's in Table 2.1 indicate the types of projects where these techniques are commonly needed. To eliminate duplication in Chapter 3, all hydrologic estimates required by all types of reservoirs are included under Sec 3.2.1 while only those additional estimates required for a certain type of reservoir are given in Sec. 3.2.2-3.2.5 (see Table 2.1). For example, a flood control reservoir would require all hydrologic estimates under 3.2.1, plus those under 3.2.2. A more detailed description of these interconnections is found in Sec. 3.1.

2.2.2 Regional Analysis for Generation of Missing Data

Regional analysis (discussed in Sec. 4.1) is needed when hydrological data, in particular historical streamflow series, are either nonexistent or too short for solving major problems in a given river basin (Klemes, 1973). The technique is based on a comparison of watershed characteristics between the one with unknown or short streamflow series and neighboring watersheds for which these series are available. Among the principal characteristics upon which the comparison is based are the sizes of the watersheds, the topography, soils and soil cover, precipitation and other meteorological factors (if available). Other factors may be introduced using the square grid technique (Solomon, 1968, and Foyster, 1975). When the homogeneity among watersheds is established, cross correlation between records can be evaluated. If this correlation is acceptable, the time series can be extended mostly by linear regression models (Benson, 1967). However, if linear regression is used to estimate a large number of values, the variance of the resulting series will be too low, and it may be necessary to use a technique that incorporates a random component (Sec. 4.1.3). Graphical techniques are also useful to detect possible interrelationships among hydrometeorological parameters in different watersheds. If no flow records are available in the problem area, runoff can be estimated using the known watershed data by simple proportion with drainage area ratios and/or rainfall ratios when they exist. These techniques are presented in more detail in Sec. 4.1.

Regional analysis is applicable to problems in which historical series are needed, for instance for reservoir sizing and nonreservoir structures for flood control, irrigation and water supply, determination of flooding potential, and for integrated river basin planning.
2.2.3 Reestablishment of Natural Flows

Most historical streamflow records reflect upstream extractions for various useful purposes. The flow values measured at these gaging stations must be corrected in order to obtain natural flows which are necessary for several hydrological analyses (Shen, 1976), such as statistical analyses (Chow, 1964), water budgets, establishment of prior water rights (legal aspects) and benefits and costs with and without proposed or existing structures (Sec. 4.2). The correction method would seem very simple, namely eliminating the effects of man-made structures such as dams. However, in many cases, information about these effects are not available and must be estimated. The problem is further complicated by the return flows from extractions that occur upstream of the gaging station and for which only rough estimates can be made. Streamflow routing procedures are often required to determine natural flows.

2.2.4 Analysis of Flow Variability

The annual flow distribution shows long-term variations in mean annual flow (Sec. 4.3). In particular, the data on dry and wet periods and trends are needed. Variations can also result from changes in land use pattern (Shiklomanov, 1975, and FAO, 1969). Time series analysis can be applied to long historical records to obtain annual flow distributions.

Intra-annual flow distribution is controlled by seasonal variations in climate (Sec. 4.3). For most purposes, a monthly distribution of flows will be sufficient and could be established on the basis of variations in meteorological parameters. For some problems, it may be necessary to consider the daily flow distribution (Sec 4.3.4). Since the occurrence of flood events is random, it is always necessary to base decisions for flood control reservoirs or other structural measures on frequency curves (Chow, 1964, and U.S. Water Resources Council, 1977). The procedures are discussed in Sec. 4.4

Low-flow frequency analysis (Sec 4.4) is also based on historical records and, in some cases, is used for determining the required conservation storage of a reservoir, a system of reservoirs, or a complete water supply system in a region. These analyses are useful for showing the necessity for low-flow augmentation.

Frequency curves of reservoir storage are used to evaluate the probability of failure to meet demands when a reservoir runs dry, or to estimate power production potential or recreational benefits (Askew, 1974b, and Askew, 1974a). The latter are based upon the frequency with which a reservoir is filled over a given level at certain time periods. Modern techniques such as queueing theory and simulation can be used to develop these storage frequency curves.

For analysis of flood protection measures, several flood characteristics are required. These include the design flood peak discharge and its corresponding recurrence interval and shape, total flood volume and time-to-peak. With the knowledge of these characteristics, the storage for flood control and the design of other structural or nonstructural measures to prevent flooding of valuable areas can be estimated for a given probability of failure. The benefits with and without (not before or after) the construction of these facilities are also based on these flood characteristics. Reservoir and channel routing procedures are used to determine inundated areas.

2.2.5 Hydrological Modelling

When historical flood data do not exist or are insufficient, the unit hydrograph or other rainfall-runoff modelling techniques can be used to estimate them using meteorological data. Several definitions of design floods are presented in Sec. 4.5. Rainfall-runoff models (see Chapter 4 - Appendix, plus Cembrowicz, 1978; Girard, 1970; and Gupta, 1977) are valuable to develop long records of flows which are needed for several of the analyses mentioned above. Their inputs are generally the rainfall and temperature measurements, evapotranspiration and soil moisture data and the watershed characteristics. This method is applied when, as often occurs, hydrological data series are short compared to the meteorological series. Another important application of these models is the short-term prediction of flows and flood forecasting. A reliable flood forecasting model (WMO, 1975a) linked with a warning system may provide a valuable nonstructural flood protection system. With these warning systems, larger damages can be prevented through evacuation of the flood prone areas or regulation of reservoirs.

Stochastic hydrology may be used to develop a number of flow records of any given length which have about the same statistical parameters as the historical record. These records are used mainly in simulation analysis to determine optimal reservoir capacity and operation.
policies. The generated records should have nearly the same means, standard deviations and serial correlation coefficients as the historical data. Nevertheless, they can include less favorable sequences of dry and wet periods or indicate that the historical record may have been an extremely unrepresentative occurrence. In applying this technique, several series of equal length should be generated and used in the simulation to obtain reliable statistical evaluations of the system's performance (WMO,1974; Pierling,1971; Yevjevich,1972; Shen,1976; Fleming,1975; and Solomon,1968).

The most widely used method for evaluating reservoir storage requirements is simulation (Schultz,1976). For flood control purposes, a short-term simulation is needed, in which the time step is much smaller than the usual monthly routings for other project purposes. Usually short interval flood studies are made for each major historical flood by simulating the operation of the reservoir projects. Regardless of the purpose of the dam, reservoir routing procedures are needed to test the ability of the spillways to evacuate the flood (Sec. 4.3). For conservation purposes, such as water supply, irrigation and hydroelectric power production, a monthly simulation analysis will also be required. For single purpose conservation reservoirs, a Rippl-diagram analysis can sometimes provide sufficient accuracy (Sec. 4.8).

Recently, more sophisticated mathematical techniques have been developed, in particular for integrated river basin planning. Among these are techniques such as optimization and simulation (Hall,1970 and Harboe,1970). These techniques may also be useful for analysis of reservoir operation problems (Sec. 4.6 and 4.9).

The objective of all these techniques is directly or indirectly an optimization of the system's parameters. Therefore, a quantitative measure of benefits and costs of each alternative design or operation must be developed. The characteristics of this system, namely mass balances, power productions, hydrologic inputs, water quality and several water demands should be developed as constraints in mathematical form. This constitutes the set of constraints under which the system's parameters to be optimized can be varied.

Mathematical models may also be used on a real-time basis to determine optimal operation of an existing reservoir or reservoir system. Such applications generally require the existence of a reliable flood forecasting system (Sec. 4.9).

2.2.6 Other Hydrologic Techniques

Sediment reserve storage is generally estimated separately (Sec. 4.8). This dead storage should be large enough so that it will not be filled before the economic life of the project is over. Sediment transport is calculated on the basis of direct measurements of bed load and suspended sediments at the reservoir site. Indirect methods based on discharge, river slope, soil cover and other parameters are also available but are less accurate. Special attention should be given to sediment transport during floods (Sec. 4.10).

Freeboard should be added to the calculated height of hydraulic structures to take account of wave action (Sec. 4.6). The amount of freeboard is partially based on experience and should be higher when damage potential is greater. It is also a function of the wind fetch.

Water surface profiles are needed to establish the height of levees for flood protection. They are based on backwater calculations or on stage-discharge curves such as are available for river gaging stations. Several techniques with different degrees of mathematical complexity have been developed and a number of computer programs are available. The selection of these techniques is treated in Sec. 4.7. Water requirements for fish and wildlife protection are estimated separately and included in storage capacity and operation studies as constraints (Sec. 4.8). For this purpose, interdisciplinary teamwork among engineers, biologists and ecologists is necessary. Recreation requirements are difficult to quantify in economic terms, which may preclude their optimization. When satisfactory economic relationships cannot be established, recreation constraints on reservoir storage levels during certain seasons or throughout the year should be developed. The final adoption of recreation criteria will usually be a matter of political process in which the affected communities express their needs, wishes and opinions about recreation.

Low flow augmentation can have several purposes as maintenance of sufficient depths for navigation, quality control through dilution of wastewaters, maintenance of fish and wildlife and recreation. Each of these purposes requires a separate study (Sec. 4.4 and 4.8).
2.2.7 Effects of Water Projects

Several positive and negative effects of all water resources projects require special attention (Sec. 4.4 and 4.10). The changed flow conditions downstream from a new reservoir can affect prior rights of other users of water. Whenever these rights exist, they should either be respected during the future operation of the reservoir or the right-holders should be compensated for their losses.

Reservoirs imply an increase in free water surface and thus in evaporation losses. These losses are higher in dry areas and where temperature and solar radiation are greater. Evaporation can be estimated on the basis of evaporation plan measurements and correction factors. This increased evaporation should be considered in determining the effective yield of the reservoir (Sec. 4.8).

Both positive and negative effects on water quality can be expected during the operation of a reservoir. Chemical and biological loads can be reduced if water stays in storage for long time periods. The dissolved oxygen (DO) content of the outflow from the reservoir can be lower than that of the inflow if water is extracted from the deep layers of the reservoir. This negative effect can be alleviated if the outlets are designed to allow extractions from the upper layers of the reservoir or if aeration facilities are provided. Several methodologies, mostly computer oriented, for analyzing the changes in water quality are presented in Sec. 4.10.4.

A reservoir reduces sediment transport since most sediment is stored in the reservoir itself (Sec. 4.10.5). If valuable sediments for agricultural purposes were previously transported during floods and deposited on cultivated areas, this will change after the construction of the reservoir which can be a disadvantage and the costs of alternative measures (e.g., fertilizers) should be studied. Degradation downstream from the reservoir may also be a serious problem.

Reservoirs may also have an effect on the ice conditions in the downstream river. The effect may be a buildup and breaking of ice dams, which could cause flooding. The ice cover on rivers and lakes may also be affected. When water from a reservoir is emptied into lakes or the ocean, the thermal and chemical properties of these waters may be changed as well. Such effects should be considered during the planning and operation of reservoirs (Sec. 4.10.8).

Of particular importance in regions where the winter season is severe, is the influence of reservoirs on the local climate. Increased winter runoff downstream of the reservoir, for example, may cause frost smoke. Such effects require special attention (Sec. 4.10.8).

Nonreservoir projects, such as levees or channel improvements, might increase the velocity of flow during floods. Drainage systems can have useful consequences through increased low flows.

Heavy floods are often caused by intensive development (e.g., urbanization). Because of the changed conditions of the terrain, a great part of the precipitation is directly transformed into surface runoff; whereas in the original state, evapotranspiration and storage in the unsaturated and saturated soil moisture zones may have taken an important part of the precipitation.

Erosion also carries large volumes of sediments into the rivers, thus causing aggradation and reduced flow capacity.

The accelerated surface runoff mechanism, together with the reduced flow capacity of the rivers, have in many cases caused severe floods. In combating such floods, the execution of partial river improvements may only lead to a shift of the problem. Basin-wide improvements should be sought, and sound land-use planning should be adopted.
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<thead>
<tr>
<th>Chapter 4 Hydrologic Techniques</th>
<th>RESERVOIRS</th>
<th>NONRESERVOIRS</th>
</tr>
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<tbody>
<tr>
<td>4.1 Regional analysis</td>
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<tr>
<td>4.2 Determination of natural flows</td>
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<td>4.3 Annual flow variability and inter-annual flow distribution</td>
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<td>4.4 Frequency curves:</td>
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<td>- Reservoir routing</td>
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<td>4.6 Determination of freeboard requirements for wind and wave action</td>
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<td>4.7 Water surface profiles</td>
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<td>- Other conservation</td>
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<td>- Sediment reserve and inactive</td>
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<td>4.10 Effects of projects on:</td>
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Table 2.1
INTERCONNECTION BETWEEN CHAPTER 3 (PROJECTS) AND CHAPTER 4 (TECHNIQUES)
References


(1) Reprints not available, copies of journals are available since 1973 at $10 to $15 U. S. dollars.
(2) Xerox copy only.
(3) May be out of print.
(4) Reprints not available, journal is available for approximately $15.
(5) Photocopies only may be ordered for $7.60.
Order From Table 1.2


(1) Available from France only.
(2) Out of print currently, may be reprinted, available at library.
(3) Cost 30 F.
(4) Reprints and journal are not available, library may have copies.
Order from USGS in Virginia, USA.

Chapter 3 is intended to provide background information on various types of water resources projects (Sec. 3.1) and to present a comprehensive list of the hydrologic estimates (as a sequence of steps) that must generally be made to design or evaluate a particular type of water resources project (Sec. 3.2-3.4). The hydrologic estimates are divided into those required for reservoir projects (Sec. 3.2), for nonreservoir projects (Sec. 3.3) and for basin-wide development (Sec. 3.4). Hydrologic estimates for reservoirs are divided into those which are applicable for all reservoir projects (Sec. 3.2.1) and the additional estimates that need to be added for specific types of projects (e.g., flood control projects (Sec. 3.2.2)).

Most of the hydrologic estimates that are listed in Chapter 3 are discussed in Chapter 4 where specific references to the details of alternative techniques and computer models are given. Table 2.1, presented in Chapter 2, provided a cross reference between the hydrologic techniques discussed and referenced in Chapter 4 and the grouping of projects adopted for Chapter 3.

The intent of Chapter 3, then, is to list in a logical order all of the hydrologic estimates needed for each project and to refer to appropriate sections of Chapter 4 for more detailed information on alternative hydrologic techniques available to make the required estimates. As indicated earlier, Chapter 4 does not contain the necessary details for using the technique, but does summarize available techniques and cites specific references where those techniques are described in detail.

Planning of water projects invariably requires many other types of computations, for example, for economic and social justification of the project. These computations are mentioned in Chapter 3 only to the extent that they relate to hydrologic inputs or are necessary to preserve the logical sequence of events needed for planning and design of water projects. The non-hydrologic steps listed in Chapter 3 are not discussed in Chapter 4 and therefore, where appropriate, specific references to available descriptions of techniques are cited in Chapter 3.

Because of the multiplicity of possible combinations of reservoir and nonreservoir projects employed in multipurpose water management schemes, all study steps prescribed herein should be interpreted in a flexible way. That is, some may not be required and individual procedural steps should be viewed as building blocks which may serve for structuring other procedures better fitted to the particular project in question.

3.1.1 Purposes and Objectives of Water Projects

One of the most visible trends in water resources projects has been the gradual broadening of their objectives. Not so long ago, water projects consisted of individual solutions to the problems which emerged in connection with some particular goal such as irrigation or hydroelectric power production. With the increasing sizes and growing number of projects, it has become evident that all the individual solutions to particular water problems often lead to conflicting situations whose compound effect on the economy is far from optimal and in many cases detrimental (United Nations, 1978).

Water resources are controlled and regulated to achieve a wide variety of objectives. Municipal and industrial water supply, irrigation, navigation improvements, hydroelectric power production, flood mitigation, and land drainage are examples of the water quantity control objectives. Pollution reduces the utility of water for municipal, agricultural, and
Industrial uses and seriously diminishes the recreational and aesthetic value of lakes and rivers, hence water quality control is also an important objective of many water projects.

Modern water projects are usually planned to simultaneously achieve several of the above-mentioned objectives, because such multipurpose projects have, as a rule, greatly improved economic justification. One classification system for water projects divides projects into either single site projects or multisite projects (either can also be single purpose or multipurpose). The single site projects are designed to serve a limited geographic area; the multisite projects consist of several spatially distributed elements in a scheme of much larger scope.

Table 3.1, based primarily on Chow(1964, pages 26-2, 26-3), illustrates the relationships among different project purposes, their objectives, and structural measures usually employed for achieving the objectives. The table indicates that the same structural measures may simultaneously serve different project purposes, and illustrates concepts underlying all multipurpose water projects.

It is quite common that the purposes of a given water project change with time. Thus, a single reservoir may be built as the first unit in an ultimate basin-wide plan and would be operated accordingly. As more reservoirs are added to the system, the method of operation of the first reservoir may be altered to complement operation of the entire system, and the original purposes of the first reservoir may change. The possibilities of altering, from time to time, the project operation rules and changing the emphasis of project purposes are features basic to all water projects.

Many objectives of water projects can also be at least partially achieved by so-called nonstructural measures. For example, the proper management of watersheds to maintain and enhance the quality of water is an important and continuous task of a predominantly nonstructural nature. Integrating water resources planning with planning for land use and for other purposes is especially important in this respect. Another example of a nonstructural measure is the regulation of flood plain use and the introduction of flood insurance policies to mitigate flood damages.

Water resources planning, therefore, requires exploring the full range of structural and nonstructural alternatives open to water resources planners. A large part of the failure to consider nonstructural alternatives in water management is attributable to institutional factors. Unfortunately, institutions responsible for planning and decision-making in the field of water resources management are often of such a character that there is little support and incentive for revealing and illuminating the nonstructural alternatives.

3.1.2 Basic Types of Water Projects

Although there are several ways of classifying water projects, in this guidebook they are basically divided into the two categories of (1) reservoir projects, and (2) nonreservoir projects. The reservoir projects include all types of storage reservoirs intended for temporal redistribution of surface flow to best meet some economic, social, and environmental purposes. Subsurface storage and groundwater recharge schemes fall outside the scope of the present guidebook. Under reservoir projects are included individual reservoirs (both single purpose and multipurpose), as well as systems of reservoirs. The reservoirs usually serve one or more of the following purposes:

a) flood control
b) hydroelectric power generation
c) municipal and industrial water supply
d) irrigation
e) navigation
f) water quality control
g) recreation
h) enhancement of fish and wildlife habitat.

In the case of multipurpose reservoirs, storage zones corresponding to one or more the above-mentioned purposes (flood control storage, conservation storage, etc.), are often designated for design and operation purposes. These allocations of reservoir storage should be interpreted in a flexible way. Sometimes, for example, in river basins where so-called flash floods occur, it may be justified to analyze flood control storage requirements separately from the conservation storage requirements. Such a procedure will lead to the determination of separate flood control and conservation storage zones. The capacity of these zones may vary over time. For example, the capacity of the flood control storage zone may sometimes be greatly reduced during seasons when flood risk is low. The determination of storage requirements for multipurpose reservoirs should therefore take into account possible trade-offs among different reservoir purposes at each particular point of time. Storage
zones (except for flood control) should not, in general, be allocated exclusively for one purpose or another (e.g., hydropower, water supply, etc.)

An individual reservoir is rarely the sole component of a water resource project. Usually, reservoirs are combined with other components, such as levees or channel improvement projects, which may serve the same purposes as the reservoirs, but do not provide for temporal redistribution of surface flow. All projects which have no storage potential are discussed in this guidebook under the category of "nonreservoir projects." The classical example of a nonreservoir project is a water supply scheme (e.g., for irrigation purposes) consisting of a long-distance water transfer from the water-excess region to the water-shortage region, resulting in a spatial redistribution of surface flow. Water transfer facilities are usually combined with reservoir projects; however, from the point of view of the hydrological computations, they present a set of problems that differs from those associated with storage reservoirs.

Typical structural and nonstructural projects for reducing flood damages include:
1) reducing peak flow by reservoirs,
2) confining the flows within a predetermined channel by levees, flood walls, or closed conduits,
3) reducing peak stages by increased velocities resulting from channel improvements,
4) diverting flood water through canals, conduits, pipelines, floodways or bypasses,
5) evacuating the flood plain,
6) floodproofing specific structures,
7) reducing flood runoff by land management,
8) flood zoning and/or flood insurance,
9) flood forecasting and warning systems.

Other types of projects which are used to provide irrigation and water supply include dams, canals, wells, distribution systems, intake works, desalting works, etc. Navigation projects include locks, dams and river stabilization works.

3.2 HYDROLOGIC ESTIMATES REQUIRED FOR RESERVOIR PROJECTS

There are several hydrologic estimates (procedural steps) of a general nature involved in the planning and design of reservoir projects irrespective of their purpose or purposes. Following the general study procedure applicable to all reservoirs (Sec. 3.2.1), some complementary (additional) procedural steps are specified in relation to flood control (Sec. 3.2.2), hydroelectric power (Sec. 3.2.3), and conservation storage (Sec. 3.2.4). A brief list of procedural steps is next offered for multipurpose reservoir projects (Sec. 3.2.5) which should be used jointly with all the preceding steps. The final section on mixes of reservoirs and other structural and nonstructural measures (Sec. 3.2.6) provides a link between the discussions of reservoir and nonreservoir projects.

3.2.1 General Steps for All Reservoirs

1) In the case of ungaged sites or sites with insufficient flow records, perform regional studies utilizing data from nearby locations. Regionalized quantities may include average annual precipitation, precipitation intensity-duration-frequency relations, unit hydrograph parameters, annual runoff, and extreme runoff-frequency relations (Sec. 4.1).
2) Determine natural flows at the reservoir and at downstream damage centers (Sec. 4.2).
3) Determine reservoir characteristics such as area/capacity/elevation curves (Chow, 1964, pages 25-65; Linsley, 1979, page 148).
4) Determine reservoir storage and pool elevation frequency curves from results of sequential routings where reservoir is operated for all authorized project purposes (Sec. 4.4 and 4.8).
5) Determine design floods for establishing reservoir spillway capacity. Determine reservoir and channel routing criteria (Sec. 4.5).
6) Determine maximum reservoir elevation by routing the spillway design flood (Sec. 4.5).
7) Determine reservoir freeboard requirements for wind and wave action (Sec. 4.6).
8) Determine water surface elevations throughout reservoir and tributaries for hypothetical dam failures and for selected design floods (Sec. 4.7).
9) Determine quantity and distribution of sediment deposited in the reservoir (sediment storage) for selected future time periods (Sec. 4.8).
10) Determine reservoir inactive storage, and minimum pool elevation to satisfy requirements for fish and wildlife, recreation, etc. (Sec. 4.8).


12) Determine reservoir capacity reduction due to ice and snow conditions, if appropriate (Sec. 4.8).

13) Determine reservoir operation rules (Sec. 4.9).

14) Determine the probability of failure of meeting the reservoir purpose(s) (Linsley, 1975, page 391; WMO, 1974a, page A20; Linsley, 1975, pages 157-158).

15) Determine effects of reservoir on streamflows, environmental quality (including temperature, DO, BOD, etc.), sediment movement within the backwater reach and below the project, downstream channel degradation, bank sloughing, evaporation, fish and wildlife, groundwater regime and ice phenomena (Sec. 4.10).

3.2.2 Complementary Steps for Reservoirs with Flood Control Storage

1) Determine reservoir operating plan considering ability to forecast future flows during flood emergency, including selection of downstream locations for which the reservoir should be operated to reduce flood damages (Sec. 4.9).

2) Establish minimum foresight period of streamflow forecasts that can influence operation of reservoirs and the corresponding average forecast accuracy for that length of forecast (for example, a 20 percent error for flows up to 12 hours in the future) (Sec. 4.9.7).

3) Determine flood control storage requirements and corresponding regulated flows for proposed reservoir by performing sequential routing studies in which flooding at major damage centers is minimized (Sec. 4.8).

4) Determine peak discharge-frequency curves for natural and regulated conditions at each major damage center below the proposed reservoir for a range of flood storage capacities (Sec. 4.4).

5) Determine outlet capacity requirements considering downstream channel capacities and desired rate of evacuation (drawdown) (Linsley, 1979, pages 240-242).

6) On the basis of stage-discharge-flood damage relationships and the peak discharge-frequency curves, determine a preproject flood damage frequency curve and average annual preproject flood damages at each major flood damage center (WMO, 1974a, page 5.70; James, 1971, pages 229-263).


8) For each flood control alternative, determine the modified flood damage frequency curve. Estimate the expected value of mean annual flood losses averted (flood control benefits) (Sec. 4.4; WMO, 1974a, page 5.70; James, 1971, pages 229-263).


10) Apply economic (cost-benefit analysis) and other criteria applicable in the given context (social, ecological, etc.) for selection of the most advisable flood control alternative. Whenever necessary, the procedure should be based not only on the single valued flood discharge-flood loss relationship but should also take into account seasonal and duration effects (James, 1971, pages 229-263).

3.2.3 Complementary Steps for Reservoirs with Hydroelectric Power Storage

1) Determine at-site energy demands which the project must meet. For power systems, determine system demands and the minimum project demands (Sec. 4.8.4; Linsley, 1979, page 470; James, 1971, pages 325-349).

2) Determine reservoir evaporation rates for normal and drought conditions (Sec. 4.10).

3) Determine reservoir inflows to be used in sequential routings under project conditions on a monthly, 10-day or weekly basis (Sec. 4.8.3 and 4.3).

4) Determine the reservoir power storage requirements and pool elevations which are necessary to provide the annual firm energy (Sec. 4.8.4).

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
5) Determine the installed capacity for the project from the regionally selected marketable plant factor and the annual firm energy (Sec. 4.8.4; Linsley, 1979, page 472; James, 1971, pages 325-349).
6) Determine the annual secondary energy and dependable capacity from long-term sequential routings (Sec. 4.8.4).
7) Based on plant factor, power schedule, and channel characteristics, determine need for re-regulating structure and/or effect of tailwater fluctuation (Linsley, 1979, pages 472-474; James, 1971, pages 325-349).

3.2.4 Complementary Steps for Reservoirs with Conservation Storage

1) Determine demands on the reservoir for conservation purposes, such as municipal and irrigation demands (generally monthly, 10-day or weekly schedules) (Linsley, 1979, pages 404-413; James, 1971, pages 285-324).
2) Determine reservoir evaporation rates for normal and drought conditions (Sec. 4.10).
3) Determine reservoir inflows to be used in sequential routings under project conditions on a monthly, 10-day or weekly basis (Sec. 4.8.3 and 4.3).
4) Determine reservoir conservation storage required to meet the demands on the reservoir (Sec. 4.8).

3.2.5 Complementary Steps for Multipurpose Reservoirs

1) Determine reservoir purposes and magnitude of demands corresponding to each purpose and their variability; determine priorities among reservoir purposes (Sec. 4.9).
2) Determine if water delivery will be direct from reservoir or if it will be diverted from a downstream point which will allow release to go through turbines (Linsley, 1979, page 678).
3) Determine need, size and elevation of multiple level outlet capacity (Sec. 4.10 and Linsley, 1979, pages 230-242).
4) Determine reservoir storage and corresponding pool elevations to accomplish multiple use of storage as determined by sequential routings for all purposes (Sec. 4.8).

3.2.6 Complementary Steps for Mixes of Reservoirs and Other Structural and Nonstructural Measures

1) Determine design floods for selected degrees of protection for reservoirs, levees, channel improvements, diversions, relocations, flood-proofing, flood plain zoning, etc. (Sec. 4.5).
2) Determine selected combination of reservoir storage, levees, channels, diversions, relocations, etc., which would produce the best plan for the whole basin considering all pertinent hydrologic, economic, and social criteria (Linsley, 1979, pages 665-680).

3.3 HYDROLOGIC ESTIMATES REQUIRED FOR NONRESERVOIR PROJECTS

The following hydrologic estimates (procedural steps) refer to specific types of water projects which do not provide for flow control by means of storage reservoirs. The steps for multipurpose nonreservoir projects are additive just like those for reservoir projects (e.g., a flood control and water quality project would consist of steps in Sec. 3.3.1 and 3.3.2). Further details on some of the specific techniques described below can be found in James and Lee (1971).

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.

**Further information on the above problems can be found in numerous references (United Nations, 1964; United Nations, 1969; United Nations, 1974; and United Nations, 1978).
3.3.1 Flood Control Projects

1) Determine preproject peak discharge-frequency curves at each major flood damage center (Sec. 4.4).
2) In the case of ungaged sites or sites with insufficient flow records, perform regional studies utilizing data from nearby stations (Sec. 4.1).
3) Determine natural flows at the reservoirs and at downstream damage centers (Sec. 4.2).
4) Determine design floods for nonreservoir structures (Sec. 4.5).
5) Determine freeboard requirements for wind and wave action for nonreservoir structures (Sec. 4.6).
6) Determine water surface elevations for length of channel or levee and beyond (Sec. 4.7).
7) Determine effects of changes on flows, environmental quality, etc. (Sec. 4.10).
8) Determine probability of failure during life of proposed structure (Linsley, 1979, pages 157-158).
9) On the basis of stage-discharge-flood damage relationships and the peak discharge-frequency curves, determine a preproject flood damage frequency curve and average annual preproject flood damage at each major flood damage center (WMO, 1974a, page 5.70; James, 1971, pages 229-262).
11) For each flood control alternative, determine the modified flood damage frequency curve. Estimate the expected value of mean annual flood losses averted (flood control benefits) (Sec. 4.4; WMO, 1974a, page 5.70; James, 1971, pages 229-262).
13) Apply economic (cost-benefits analysis) and other criteria applicable in the given context (social, ecological, etc.) for selection of the most advisable flood control alternative. Whenever possible, the procedure should be based not only on the single valued flood discharge-flood loss relationship but should also take into account seasonal and duration effects. (Chow, 1964, pages 25.116-25.120; James, 1971, pages 229 262).

3.3.2 Water Quality Control Projects

1) Perform regional studies to locate all present and potential water uses that affect or are affected by water quality (Chow, 1964, pages 19-23). The major water uses of interest are:
   a) municipal, including storm water runoff,
   b) industrial,
   c) agricultural,
   d) feedlot operations,
   e) commercial fishing,
   f) water contact recreation, and
   g) non-water contact recreation.
2) For each of the present and potential water uses, determine (Chow, 1964, pages 19-27; Thomann, 1972):
   a) intake water quality requirements,
   b) intake water treatment alternatives (technical and economic data),
   c) wastewater discharge loads in relation to different water use technologies, and
temporal distribution of wastewater discharges, and
   d) wastewater treatment alternatives (technical and economic data).
3) Perform hydrologic studies to determine design streamflow rates for further analysis of waste-assimilative capacity of the river system (Sec. 4.4). These studies should be concerned with:
   a) duration of low-flow periods and ice-covered periods
   b) possibility of occurrence of low flows of selected duration.

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
c) severity (e.g., total deficiency in water supply with respect to some reference flow and some duration),
d) time of occurrence within the annual cycle, and
e) areal extent of low-flow phenomena.

4) Perform waste-assimilative studies for the given river system (including estimation of dispersion and mixing zones), for different design (reference) and streamflow rates (see Sec. 4.10).

5) Perform water quality management studies to determine the most desirable (e.g., least costly) way of achieving specific water quality objectives (James, 1971, and Thomann, 1972).

3.3.3 Irrigation Projects


2) Determine irrigation requirements (the amount of water that should be supplied to 1 hectare of a field during the irrigation period to create conditions most favorable for the crop growth) (WMO, 1974a, page A.7; Doneen, 1971b; and Withers, 1974).

3) Determine water intake needs on the basis of irrigation requirements (with due consideration given to their seasonal variability), irrigation area, crop rotation, and the efficiency of the future irrigation system (WMO, 1974a, page A.7; Doneen, 1971b; and Zimmerman, 1966).

4) On the basis of historical flow records (or maps of runoff isolines) compute long-term normal flow which is to be used for determining irrigation capacity of the stream (Sec. 4.2 and 4.3).

5) Analyze intra-annual flow distribution. Identify critical flow values and critical seasons of the year (Sec. 4.3).

6) Perform flood flow analysis for determining the size of the flow control structures, storm water inlets, mudflow channels, and flood gates in order to avoid destruction of hydraulic structures and erosion of the irrigation network (Sec. 4.5 and Houk, 1956).

7) Perform analysis of water level (stage) variability in order to determine location, type, and size of irrigation water intake structures (Sec. 4.3).

8) Perform analysis of thermal and ice regime prevailing in a given area, in order to determine the operation rules for hydraulic structures, and to determine heating requirements for gates and gates (Sec. 4.10).

9) Compute water sediment concentration and its variability with respect to streamflow rate, stream depth, width and length (Sec. 4.10).

10) Analyze chemical composition of irrigation water during different seasons of a year (high flows, low flows, etc) (Chow, 1964, page 19-1; Ayers, 1976).

11) Perform water-balance computations based on the surface water measurements of inflow, outflow, evaporation, ground water regime, moisture dynamics in the aeration zone, meteorological conditions, vegetation state and its growth (Sec. 4.1).

12) Estimate the volume of irrigation runoff, return flow (see Sec. 3.3.4) and evapotranspiration losses; analyze the possibilities of irrigation runoff reuse for other purposes (WMO, 1974a, page A.7; Doneen, 1971b).

13) Assess the environmental impact of the contemplated irrigation project (soil salinization, water-logging, water quality deterioration, streamflow reduction, etc.) (Sec. 4.10).

3.3.4 Drainage Projects

1) Carry out regional study of rivers in the regions of future drainage of swamps and marsh-ridden areas in order to estimate their water-carrying capacity during spring-flood and rainfall-flood periods (Sec. 4.7).

2) Investigate areas to be drained and arrange for hydrologic data collection.

3) Investigate river basins with existing drainage projects to obtain (Chow, 1964, pages 21-88; Luthin, 1966):
   a) detailed description of drainage systems,
   b) present use of drained lands,

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
c) the state of drainage canals and rivers,
d) the character of man's activity within the watershed, and
e) description of the state of lands adjacent to drainage systems within natural landscapes.

4) Investigate hydrological regime of reclaimed areas in order to develop regional estimates for hydrological computation of drainage systems, namely (Sec. 4.1 and WMO,1974a, pp. 5.64-5.72):
   a) mean flow,
   b) maximum and minimum discharges during various seasons
   c) local runoff,

5) Investigate influence of reclamation works on water and heat regime, water resources and water content of reclaimed and adjacent areas, including (WMO,1974a, page 5.86; Doneen,1971a; Doneen,1973, and Kienitz,1979):
   a) moisture exchange within the aeration zone,
   b) water level regime,
   c) flow regime,
   d) evaporation,
   e) infiltration of precipitation and waters for irrigation from reclamation systems,
   f) moisture content of soils and subsoils,
   g) regime of water use by different kinds of crops,
   h) water quality.

6) Develop regional recommendations for purposeful management of water, heat, air and feeding regimes within the zone of active water exchange, and first of all of the active zone with roots layer, of under reclamation (Chow, 1964; Doneen,1971a, and Doneen,1973).

7) Investigate the influence of reclamation on natural landscapes in different physiographical and climatic conditions in order to evaluate the character of their change and change of biological productivity (Chow,1964, pages 21-27; Doneen,1971a, and Doneen,1973).

8) Organize collection of necessary observational data to provide hydrometeorological information for reclamation systems in order to increase their efficiency and the productivity of drained lands (Chow,1964, page 21-4; and WMO,1974a, page 3.1; Doneen,1971a, and Doneen 1973).

3.3.5 Municipal and Industrial Water Supply Projects

1) Forecast alternative levels of water demand by industry and municipalities (Linsley, 1979, page 406; James,1971).*

2) Perform regional studies to define location of potential sources of water supply (surface and groundwater resources) (Sec. 4.1 and WMO,1974a, page A.3).

3) For each potential supply source, determine (Sec. 4.3 and 4.4):
   a) volume and quality of water available as related to different water supply reliability levels,
   b) time-distribution of available water resources, both in terms of their quantity and quality, and
   c) cost of resource development alternatives.

4) Perform demand/supply analysis to select the most desirable (e.g., least costly) supply scheme (James,1971)*.

5) Determine design parameters of diversion and intake facilities (e.g., head-gates, pumps, compensation reservoirs) (Linsley,1979, pages 455-461)*.

6) Determine design parameters of water treatment (purification) plants, if necessary (Linsley,1979, pages 429-431; Thomann,1972)*.

7) Determine design parameters of the delivery system (Linsley,1979; page 297)*.

8) Determine effects of proposed solution on basin-wide flows, water quality, sediment movement, etc.(Chow,1964, pp. 14-1, 17-1, and 19-1; Ortolano,19_)*.

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
3.3.6 Navigation Projects

1) Forecast alternative levels of freight to be transported by inland navigation versus other means of transportation (Linsley, 1979, page 486; James, 1971, pp. 353-372)*.
2) Determine transit depths and channel conditions necessary for a given type of barge (Sec. 4.2 and 4.4).
3) Determine depth-duration curves for natural flow conditions (Sec. 4.4). If natural flow conditions do not allow sufficiently reliable transit depths to be maintained throughout the navigation season, identify potential depth control alternatives such as channel improvements, low-flow augmentation by controlled reservoir releases, and/or water impoundment by navigation weirs and dams (river canalization) (Linsley, 1979, page 488; Fair and Geyer, 1968).
4) Perform hydrological analysis indicating in probabilistic terms, the extent to which each depth and channel control alternative improves navigation conditions (Sec. 4.4).
5) Analyze navigation conditions during high water levels (clearance heights under the bridges, flow turbulence, etc.). Determine maximum allowable water levels for navigation (Linsley, 1979, page 489).
6) Evaluate the effects of tides, winds, and ice phenomena on navigation conditions (Sec. 4.6 and 4.10).
7) Evaluate the sediment transport regime and the risk of lock silting (Sec. 4.10).
8) In case of artificial navigation waterways, estimate flow diversion requirements and characteristic discharges for design of diversion structures (Sec. 4.5).
9) Estimate characteristic water levels for design of harbour facilities (Wiegel, 1964, page 442; Quinn, 1961)*.

3.3.7 Recreation and Fish and Wildlife Projects

1) Distinguish between water-oriented recreation projects and projects for nature conservation (James, 1971, pages 395 and 436).
2) Select areas that are feasible for one or both purposes (James, 1971, pages 396 and 423).
3) Perform a general geographic, hydrologic and ecological study for the area under consideration (United Nations, 1964, page 4; Chow, 1964, pp. 26-4 and 26-13).
4) Investigate potential recreation uses for the area (James, 1971, page 398; Clawson, 1966)*.
5) Determine the consequences of intensive recreation; analyze possible restrictions against excessive use (Sec. 4.10 and James, 1971, page 400).*
6) Select areas for mass recreation and for individual recreation; allocate land to various alternate recreation activities (James, 1971, page 398; Clawson, 1966)*.
7) With regard to nature conservation, prepare provisions against excessive accessibility and inappropriate land use (United Nations, 1970, page 72; Clawson, 1966)*.

3.4 HYDROLOGIC ESTIMATES REQUIRED FOR BASIN-WIDE LONG-TERM PLANNING FOR INTEGRATED DEVELOPMENT OF WATER RESOURCES

The integrated development of a river basin involves a multiplicity of studies concerned with both the physical and socioeconomic environment of man. This section is concerned chiefly with the hydrologic studies required in the preparation of a basin-wide plan for integrated development of water resources. Although hydrologic studies are of fundamental importance for preparation of such a plan, the overall success of the planning effort depends upon critical analysis of all alternative means of promoting social welfare and economic growth in the river basin. The development of water resources cannot be considered an end in itself, but rather as a contribution to the general progress as envisioned by the local authorities and the national government.

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
There are nearly as many study procedures for integrated basin-wide development as there have been river basin projects. The following specification, based on United Nations (1970), serves primarily as an illustrative example and also as a checklist of the major steps in hydrologic investigations for basin-wide planning concerning integrated development of water resources.

1) Appraise the adequacy of available hydrometeorological data (e.g., precipitation, evaporation, evapotranspiration, air temperature) and hydrologic data (e.g., flow time series, flood hydrographs, aquifer recharge characteristics, quality parameters, sediment transport parameters) (Sec. 4.1 and 4.2).

2) Select key control points in the river system taking into consideration location of stream-gaging stations, major water users, present and planned flow control structures. Define location of groundwater supply sources (United Nations, 1964, page 3).

3) Determine what additional data are required with consideration given to the purpose of the investigations and the methods which are to be used for subsequent water management analysis (e.g., classical balances, simulation, optimization) (United Nations, 1970, page 47).

4) Devise methods, standards, and schedules for acquiring additional data (e.g., extension of flow records, application of rainfall-runoff models, regional analysis) (Sec. 4.1 and 4.2).


6) Analyze and organize the data for studying problems of (United Nations, 1970; Fair and Geyer, 1968):
   - municipal, industrial, and agricultural water supply,
   - flood control,
   - water quality control,
   - hydroelectric power generation,
   - inland navigation,
   - recreational use of water, and
   - nature conservation.

7) Estimate annual flow variability and the characteristics of intra-annual flow distribution (Sec. 4.3).

8) Estimate the amount of water which can be withdrawn from groundwater resources without producing undesired results (Linsley, 1979, page 101; Bouver, 1978).

9) Estimate the minimum flow requirements in the control profiles (i.e., flow which must be maintained because of aesthetic, scenic, sanitary, and/or biological reasons) (Chow, 1964, pages 19-27; Linsley, 1975).


11) Estimate flood characteristics for selected profiles (e.g., peak flow frequency curves for natural and regulated flow, design flood flows, channel routing criteria) (Sec. 4.2, 4.4 and 4.5).


13) Estimate low-flow characteristics for selected profiles (e.g., minimum flow frequency curves for natural and regulated flow, design low flows) (Sec. 4.4).

14) Develop water quality control alternatives (e.g., low flow augmentation) (Chow, 1964, pages 18-22; Eckenfelder, 1970).


16) Develop hydroelectric power generation alternatives (Linsley, 1979, page 467; Chow, 1964, page 26-4; Creager, 1958)*.

17) Estimate hydrologic characteristics for inland navigation studies (e.g., depth, width, current velocity) (Chow, 1964, page 26-2; WHO, 1974a, page 11.35; Stratton, 1969).

18) Develop navigation development alternatives (Chow, 1964, page 26-2; Linsley, 1979, page 677; Stratton, 1969)*.

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
19) Estimate hydrologic characteristics for recreational studies (e.g., depth, area of water surface, water quality (James, 1971, page 408; Chow, 1964, page 26-2; Linsley, 1979, page 677; Clawson, 1966).

20) Develop alternatives for recreational use of water (James, 1971, page 408; Chow, 1964, page 26-2; Linsley, 1979, page 677; Clawson, 1966)*.

21) Estimate hydrologic characteristics for fish and wildlife studies (e.g., water temperature, water quality) (James, 1971, page 423; Linsley, 1979, page 241; Chow, 1964, page 26-2).

22) Develop alternatives of habitat improvements for fish and wildlife (Chow, 1964, page 26-2; James, 1971, page 423; Linsley, 1979, page 241).*


24) Evaluate all alternatives and prepare a comprehensive long-term plan for the integrated development of water resources in the river basin, including an assessment of impacts of projects on the environment and hydrologic regime (Sec. 4.10)*.

*This procedural step is not covered in Chapter 4 since it is outside of the hydrologic engineering area.
<table>
<thead>
<tr>
<th>PROJECT PURPOSE</th>
<th>OBJECTIVES</th>
<th>STRUCTURAL MEASURES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flood Control</td>
<td>Flood-damage prevention or reduction, protection of life and economic development</td>
<td>Dams, storage reservoirs, levees, floodwalls, channel improvements, floodways, pumping stations, flood warning systems, diversions and other flow retarding measures</td>
</tr>
<tr>
<td>Hydroelectric Power Generation</td>
<td>Provision of electric power for economic development and improving living standards</td>
<td>Dams, storage reservoirs, penstocks, power plants</td>
</tr>
<tr>
<td>Municipal and Industrial Water Supply</td>
<td>Provision of water for municipal and industrial uses</td>
<td>Dams, storage reservoirs, wells, conduits, pumping plants, intake works, water treatment plants, saline-water conversion, distribution systems</td>
</tr>
<tr>
<td>Irrigation</td>
<td>Increase and stabilization of agricultural production</td>
<td>Dams, storage reservoirs, wells, canals, pumping stations, weed control and desilting works, distribution systems</td>
</tr>
<tr>
<td>Drainage</td>
<td>Increase and stabilization of agricultural production, urban development, protection of public health</td>
<td>Ditches, tile drains, pumping stations, sluices</td>
</tr>
<tr>
<td>Navigation</td>
<td>Transportation of goods and passengers</td>
<td>Dams, storage reservoirs, canals, locks, channel improvements, harbour works</td>
</tr>
<tr>
<td>Water Quality Control</td>
<td>Protection or improvement of water supplies for municipal, industrial and agricultural use, protection of fish and wildlife, development of commercial fishing</td>
<td>Waste treatment facilities, reservoir storage for low-flow augmentation, waste-water collection systems</td>
</tr>
<tr>
<td>Recreation</td>
<td>Enhancement of recreation and sport opportunities</td>
<td>Storage reservoirs, facilities for recreational use, pollution control works</td>
</tr>
<tr>
<td>Fish and Wildlife Enhancement</td>
<td>Improvement of habitat for fish and wildlife, reduction or prevention of fish and wildlife losses associated with men's activities, provision for expansion of commercial fishing</td>
<td>Fish hatcheries, fish ladders and screens, reservoir storage, pollution control works</td>
</tr>
<tr>
<td>Sediment Control</td>
<td>Reduction and control of silt load in streams and protection of reservoirs</td>
<td>Desilting works, channel and revetment works, bank stabilization, special dam construction</td>
</tr>
</tbody>
</table>

References


(1) Out of print, library may have copies.


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(1) Out of print, library may have copies.

(2) Available at approximately $520 U. S. dollars plus 5% postage.

4 Hydrological techniques available for water projects

4.1 TECHNIQUES FOR REGIONAL ANALYSIS OF HYDROLOGICAL AND METEOROLOGICAL DATA

4.1.1 Introduction

Section 2.1 has briefly described various procedures for analyzing hydrological and meteorological data. A statistical parameter computed for a given record, for example, the average annual flow rate, is applicable only to one site. Regional analysis is used to transfer information to ungaged or inadequately gaged sites. Examples of information that can be regionalized are: average annual precipitation, precipitation intensity-duration-frequency relations, unit hydrograph parameters, annual runoff, extreme runoff-frequency relations, etc.

4.1.2 Homogeneity of Data

Before any extensive effort is expended on regionalization, the data should be checked for temporal and spatial homogeneity. Temporal homogeneity (stationarity) is checked to detect trends or sudden changes in the data. The average value of a variable may be increasing or decreasing with time because of long cycle climatic, land use or other changes in the region as discussed by IAHS/WMO (1974). Such changes may be either natural or man-made. The method of moving averages is commonly used to determine if a trend exists (See Chow, 1964, pages 8-81 to 8-82). Sudden changes in a response function or inconsistencies in a gaged record can often be discriminated by double mass curve analysis (WMO, 1974, page 5.3; Chow, 1964, pages 9-26 to 9-27, and Unesco/WMO, 1977, page 31). Apparent nonhomogeneities of the data should be substantiated by historical evidence before being accepted as fact.

Spatial homogeneity is checked to determine if the variation in the parameter is significant and can be attributed to differing physiographic and climatic features over the region being analyzed. A homogeneity test has been developed for the Type I extremal distribution (Chow, 1964, page 8-37). This test provides a means for separating data sets into categories representative of hydrologically and meteorologically similar areas. The analyst should also be aware of different physiographic features which may influence the magnitude of a variable. For example, mountain ranges cause sudden changes in the precipitation patterns, and different soil permeability rates can affect runoff rates. An area classified as homogeneous should contain a sufficient number of stations to obtain meaningful regional relationships (WMO, 1972, Section III-3.3). A number of frequently applied methods for temporal and spatial homogeneity, as well as hydrological statistical methods, are described by Yevjevich (1972a).

Both hydrological and meteorological data are usually correlated in time and space. This correlation leads to a narrower confidence interval associated with the homogeneity criterion and is not taken into account in the theoretical distributions of sampling errors. Conversely, the existence of serial correlation (e.g., of streamflows), broadens the confidence interval associated with the homogeneity criterion (Rodhestvensky, 1974, pages 184-226, and Yevjevich, 1972b, pages 246-247).

4.1.3 Statistical Parameters

With the availability of long-term hydrometeorological observations, the methods of statistical analysis of the basic data with analytical functions of the probability distribution become feasible. Probability distributions in widespread use include: Pearson
Type III distribution (continuous binomial law), three-parameter gamma-distribution of S. N. Kritsky and M. F. Menkel, log-normal distribution, normal distribution, Gumbel distribution, exponential distribution, family of distributions of Johnson, and others (Sokolov, 1976, Chapter 2 and Klemes, 1973, pages 13-20). Parameters of the distributions (e.g., mean, standard deviation, coefficient of variation) are established from observed data (Karteshivili, 1967) and are subject to regional analysis (HEC, 1972*, Chapter 4; Rozhdestvensky, 1974, pages 21-147; Gidrometeoizdat, 1972, pages 4-8; and Gidrometeoizdat, 1973, pages 9-16 and 29-32).

Since hydrometeorological data are often rather limited, the sample parameters of the selected distribution are known to have random errors which should be taken into account in the mapping of the parameters and in the derivation of multiple linear regression coefficients to define these parameters (Rozhdestvensky, 1977, pages 83-137; and Heras, 1972).

4.1.4 Hydrometeorological Mapping

Maps of various hydrometeorological variables are often prepared because a large amount of information can be efficiently displayed in this manner. Also, such displays are easily understood by nontechnical persons. Unesco/WMO (1977) contains many examples for displaying atmospheric, surface and groundwater characteristics.

The hydrologist should first identify the primary purpose and intended use of a map, then prepare the map to show the desired information in a clear form. If maps of several variables are prepared for the same area, they should be based on the same period of record so that comparisons are meaningful. The mapping of hydrological characteristics and parameters of hydrometeorological data requires consideration of the topography, subsurface structures, and the nature of atmospheric phenomena. In addition to the zonal factors, there are local factors, such as undrained depressions, river network density, watershed slope, drainage area, karstic features, character of the underlying surface, etc., which should be considered when mapping specific characteristics. The effect of local factors on runoff characteristics tends to decrease with increasing drainage area. Beyond some size of drainage area, areal changes of the runoff characteristics tend to reflect mostly the geographical zonality and are only slightly dependent on local factors. Meteorological elements are usually less dependent on the zonal factors of the surface and subsurface characteristics as compared to hydrological characteristics and associated parameters (Rozhdestvensky, 1977).

Mapping with isolines of the hydrological or meteorological data, or some statistical parameter of the data, is a common way of displaying information. Although the preparation of isolines may appear to be simple, great care is needed especially where the density of the measured points is sparse and/or there are large changes in the geologic or climatic conditions. In the preparation of isoline maps, the degree of smoothing should be consistent with the sampling errors of the stations. Too little smoothing could indicate regional variation, whereas the true cause may be sampling variability (WMO, 1974, pages 5.5 and 5.34; and Unesco/WMO, 1977, pages 198-199).

Application of the square grid technique provides a means of storing, processing and retrieving information by computer. This technique consists of dividing the area into uniform squares; grid intervals between 1 and 10 km are usually adequate. Then, various characteristics of each square grid are recorded. This technique has been applied to precipitation, temperature, runoff and watershed characteristics (WMO, 1972, Section III-2.1).

4.1.5 Regression Analysis

Often it is possible to develop relations with climatic and basin characteristics to define selected hydrologic variables. The acceptability of the derived relationship will depend upon the computed measure of fit (commonly, the standard error of estimate). A graphical representation of these usually multivariable dependencies is rather complicated. Therefore, to describe relations between hydrological characteristics and the causative factors, multiple linear regression analysis is generally used (Chow, 1964, Section 8-II; HEC, 1972, Chapter 4; and Rozhdestvensky, 1974, pages 318-347). In case of a nonlinear relationship, a linearization is usually possible by the use of an appropriate transformation. In many cases, it is often quite sufficient to make a logarithmic transformation of the variables involved.

*The abbreviation HEC is used throughout the text in order to briefly describe reference to publications from the Hydrologic Engineering Center of the United States Army Corps of Engineers.
The relations developed by use of multiple linear regression can be used for the extension of a short hydrological series to a series representative of a longer period, for the spatial interpolation of hydrological characteristics, for the estimation of statistical streamflow parameters, and for establishing the relationship between runoff and causative factors (Rozhdestvensky, 1974, pages 338-347; WMO, 1974, pages 5.53-5.55).

A difficulty arising in the use of multiple linear regression is the evaluation of the stability of the derived equations and the reliability of the regression coefficients. Experience in the use of multiple linear regression equations shows that the use of a large number of the so-called independent variables leads, as a rule, to instability in the coefficients. The number of independent variables in a majority of cases should not exceed three or four since a further increase in the number of variables makes the system of equations unstable for determining the regression coefficients and can lead to large random errors in the computation of regression coefficients. The selection and use of multiple linear regression equations must be done with extreme care. The independent variables in the final equations should have known physical relationships with the dependent variable. Also, values for the independent variables at other locations should be easily acquired. The regression coefficients should have the correct sign, e.g., an increase in precipitation should cause an increase in runoff. It is not valid to change one of the independent variables and observe the change in the dependent variable, as there are usually important intercorrelations between the independent variables which must be taken into account. Also, these equations should not be used with variable magnitudes outside the range of those in the regression analysis data set. It is wise to confirm the potential equations with other specialists or sources, and by validation through split sampling.

The numerical computations for the solution of multiple linear regression equations are usually made with the use of computers. Many programs automatically select the most effective independent variables in a step-wise procedure by applying a criterion for the addition or deletion of a variable. Regression analysis techniques are described in Chow (1964) (Section 8-II) and HEC (1972) (Chapter 4).

4.1.6 Water Balance Ratios

Water balance ratios may be regionalized for an area under study. The selection of the intervals of time and space for averaging the elements composing a water balance equation is a function of the amount of hydrometeorological knowledge available for the study area, as well as by the reliability and accuracy of methods for the computation of water balance elements at sites with inadequate observational data. A major difficulty with the derivation of water balance equations is the evaluation of the accuracy of the individual components and of the balance as a whole. Without such an evaluation, the use of this method in the regional analysis of hydrological information may be of little value (Unesco/WMO, 1977, page 35; Sokolov 1974).

4.1.7 Application to Inadequately Gaged Sites

Records at some stations may be very short and, therefore, have a large time-sampling error. Data at long-record stations can be correlated with the short-record station to assist in providing a better estimate at the short-record station. If only a few values are missing, the values could be estimated by linear regression, but estimation of many values will cause a reduction in the variation of the values at the short-record station. There are adjustment equations that can be used to adjust the mean and standard deviation of a short-record station with a long-record station (WMO, 1974, pages 5.53-5.55). A threshold level of the correlation should be exceeded for the adjustments to be appropriate.

The characteristic computed at a short-record station can be weighted with the results of a regional study. Application of this procedure requires computing the worth of the regional information in terms of equivalent record length or the inverse of the error of variance. The weighted characteristic is then computed by weighting each characteristic (regional and short-record station) by its record length or inverse of the error of variance.
References


*Major English language reference.

(1) Out of print, new edition foreseen in 1982 (the cost will be approximately $100).
(2) Order from IAHS in France.
(3) Out of print Oct 1980 (may be reprinted), see libraries.
(4) Out of print; may be available in libraries.


4.2. DETERMINATION OF NATURAL FLOWS

4.2.1 Introduction

The term "natural flows" has several meanings. In a strictly water supply or basin water yield sense, it might refer to the amount of daily, monthly or yearly runoff passing a specified data collection point, assuming the area contributing to runoff was in its native or "natural" condition. Since this condition is seldom attainable, a more liberal definition might refer to observed runoff adjusted for withdrawals and consumptive use by municipal, industrial, or agricultural purposes and for any inter-basin water transfer. This second definition is more practical and is generally what the user means when discussing water supply yield, water rights, and associated studies. A third usage of the term "natural flow," commonly used in a flood control sense, refers to a "preproject" condition, or the flow that would result from a specified storm event prior to any man-made storage, levee, channel or flood plain encroachment structures. Subsequent discussion in this subchapter will outline techniques and considerations applicable to definitions two and three.

4.2.2 Data Inventory

Before any water supply yield study can be accomplished, an inventory must be conducted to determine the nature, magnitude and distribution of present water use by man, both upstream and downstream of any proposed project sites. (Chow,1964, Section 26; Linsley,1972, Chapter 21; Linsley,1975, Chapter 4; Bureau of Reclamation,1977, Chapter 1, Section 10; WMO,1958, Section 5, HEC,1977, Chapter 3; and Grushevsky,1969). Data sources would include annual publications by national, state and local water resources inventory and regulating agencies as well as unpublished data collected by project operating agencies. Site specific data often will be needed at ungaged sites. This will require studies of runoff correlation with physical parameters such as drainage area shape, size, elevation, length and slope and climatic parameters such as temperature, snowfall, and rainfall. These regression techniques are discussed in subchapter 4.1.

It is often necessary to fill in missing data periods by correlation and stochastic procedures (HEC,1972, Chapter 5; WMO,1974, page 5.5; Gidrometeoizdat,1973, pages 338-348) in order to develop a common base period for analysis which reflects high and low flow extremes as well as being representative of long-term averages. Next, an inventory of water withdrawals and imports, lake storage changes, and lake evaporation estimates are required before a water budget or sequential balance can be accomplished. In some countries, present water usage cannot be curtailed without prior consent and appropriate reimbursement or financial settlement to the affected landowner. Many areas of the world have existing water laws, often very complex, which govern the use of surface water resources (Chow,1964, Section 27; Linsley,1972, Chapter 6). In some cases, regulations cover groundwater interrelationships. It is not the purpose of this discussion to cover the many aspects of these legal complexities, but merely to point out that hydrologists must be aware of applicable legal responsibilities when considering the alteration of the flow regime of river systems. In any economic evaluation of the benefits or damage resulting from proposed structural measures, a plane of reference called "natural flow" or "preproject flow" must be well defined. Proof is required that a benefit claimed for a reservoir release is truly a result of flow that would not have existed without the project. Similarly, if releases are curtailed during flood periods (i.e., periods of storage accumulation), evidence must be presented to determine the magnitude of the benefits (Neshikovsky,1971; Grushevsky,1969; and Zheleznyak,1963).

4.2.3 Rainfall-Runoff Relations

Some areas of the world have a very limited amount of recorded runoff data, yet rainfall data may be more available. From available rainfall data, regional generalization of seasonal and annual amounts can be made and displayed on rainfall maps. By correlation of monthly and annual rainfall amounts with available runoff data, often a more reliable estimate of high values of streamflow can be made than if based on streamflow data alone (WMO,1974, Chapters 5.3 and 5.5). There is usually some correlation between successive months of runoff (serial correlation) which must be included in making estimates from rainfall. Also, snowpack and air temperature are typical parameters included when making runoff estimates (WMO,1974, Chapter 6.4). Rainfall-runoff models are discussed in more detail in Chapter 4.5 and in the Appendix to this chapter.
4.2.4. Reservoir Routing

Typically, in water supply, recreation, and hydropower storage projects, water balancing or budgeting (often called "reservoir routing") studies must be conducted for critical low flow periods of the historical record to evaluate the consequences of a proposed set of regulating rules and to determine preproject flow estimates for existing projects (Linsley, 1972, Chapter 7; HEC, 1975, Chapter 6; Klemes, 1973, Chapter 3; and Heras, 1976, pages 51-70). In many countries, a number of computer programs (such as HEC-5 (HEC, 1977), and HEC-5 (HEC, 1979) in the United States), have been developed by various private and governmental groups for analyzing the project yield under a specified set of operation rules and water demands. Some of these generalized models include capabilities for economic analysis, multipurpose usage, and multireservoir analysis. This project analysis is often done using a 1-month time interval, but in the case of hydroelectric energy generation, a weekly or daily time interval during critically low flow periods may be required. The analysis may cover a total time period of, say, 2 to 20 critically low flow years, or the complete period of observed or synthesized record. The basis for any claimed benefit, however, must be an improvement over the natural or preproject conditions (Shiklomanov, 1979, pages 7-82 and 264-285).

4.2.5. Channel Routing

In the specific case of flood control projects, peak stage or discharges, or a set of observed or synthesized stage/flow hydrographs are usually the basis of comparing consequent benefits or damage rather than mean monthly or annual flow. The reasons for this are obvious, since generally flood damage is directly proportional to flood stage. The season of the year and the duration of flooding are sometimes included as parameters affecting the extent of damage or claimed damage reduction benefits. Techniques used to estimate these "preproject" or natural flood peaks at downstream damage centers after a project has been constructed vary from very simple short interval averaging and lagging of flows to complex unsteady flow equation analysis in one or two dimensions. In reservoir storage projects, the difference in inflow and outflow (storage change) during each time interval is computed or estimated from gaged flow data or from known outflow and storage changes (Kuchment, 1972). If linear channel routing techniques such as the Muskingum (Linsley, 1975, paragraph 9-8) or the Kalinin and Miljukov methods (WHO, 1974, page 5.63) are reasonably applicable, these reservoir storage changes (expressed in units of flow) can be routed to successive downstream damage centers or index points and added to observed or estimated discharges in the proper time sequence to determine the maximum preproject or "natural" peak discharge. (See Chow, 1974, Section 25-II; Linsley, 1975, Chapter 9; WHO, 1974, Pages 5.61-5.64, and HEC, 1979). Peak discharge can then be converted to stage or elevation and related to damage to determine the comparative damage under the two conditions. After a number of such observed and/or hypothetical events have been analyzed in this manner over a broad range in magnitude, a discharge reduction curve can be constructed for each index location where discharge-frequency or stage-frequency curves have been determined for either preproject or project conditions. The two variables plotted on such a curve are peak discharge with and without the project in operation. These reduction curves can then be used to modify the discharge or stage-frequency curves to the desired preproject or project condition and average annual damage estimated by procedures discussed elsewhere in this guide. Several computer programs include subroutines for making the preproject computations and damage evaluations (for example see HEC, 1979; Kuchment, 1972; Shiklomanov, 1979).

If channel and flood plain characteristics are such that one of the linear routing techniques is not reasonably applicable, one of several available nonlinear routing techniques must be applied. These would include: (a) modified Puls, (b) storage routing as used in the SSARE model (WHO, 1974, page 5.63), (c) reach index storage versus index discharge (working R and D), (Chow, 1964, Section 25-II), and (d) "hydraulic" unsteady flow methods (Linsley, 1975, Section 9-2; and Rosenberg, 1972). The first three nonlinear methods are complicated by the requirement of local intervening flow determination. The local runoff must be added to the routed reservoir (storage changes) at the head of each designated channel reach before the channel routing is performed, since the time lag and attenuation is dependent on the magnitude of total flow. A determination of the linearity or nonlinearity of the storage vs. discharge characteristics of a channel reach can be made best from observed flood events, but can also be estimated from water surface profile calculations for a range of discharges (Linsley, 1975, pages 295-297).
4.2.6 Graphic and Regression Techniques

In most countries of the world, graphic techniques and mathematically derived regression analyses are used in calculating some of the preliminary estimates of natural flows and during some regional planning phases of studies (Shiklomanov, 1979, pages 47-82). Methods used include:

(a) single and double mass curves and rating difference curves  
(b) analogy method  
(c) multiple correlation of flow related to flow forming factor  
(d) physicomathematical simulation of flow.

Mass curve analysis is discussed in paragraph 4.1.2. In the simple mass curve evaluation of annual or seasonal water discharges, the annual or seasonal total runoff is accumulated each period for the entire period of record. Any point of inflection indicates the year of possible hydrological regime change. The magnitude of such change is indicated by the departures of the plotted curve from an extension of the curve based on its slope prior to the break point. Care must be exercised to be able to rationally justify the apparent change in runoff characteristics. The double mass curve approach is similar, except the accumulated flow from a basin of known similar runoff characteristics and known unchanged hydrologic regime is used, as the abscissa rather than time (WMO, 1974, pages 5.48-5.49).

Rating difference mass curves are similar to the single curve technique and are used to reflect the changes resulting from channel modifications. The analogy method, or simple correlation, establishes some logical correlation between flow of two hydrologic similar basins, one of which has not been disturbed or altered by major physiographic or hydrologic change.

The multiple correlation method is an extension of the simple correlation. In this case, several basins could be used, or more typically, parameters such as annual or seasonal precipitation, drainage area, percent of urbanization, percent of forested area, soil type, water surface area, land slope, snow cover, temperature, etc., are correlated with runoff (see paragraph 4.1.5 and Chow, 1964, Section 8-II).

Methods of physicomathematical modeling are based on the use of basic laws of moisture and heat transfer, laws of surface and subsurface water flow, evapotranspiration, infiltration, etc. However, except for major man-made alterations such as channel modifications, reservoir storage, diversions and drainage networks, models are difficult to calibrate with a high degree of reliability because of the complexity of the hydrologic regime in nature (WMO, 1974, pages 5.94-5.95).
References


Order From Table 1.2

A-10 (1)

A-10 (2)

B-68

(1) Out of print, see libraries.
4.3 TECHNIQUES FOR EVALUATING ANNUAL FLOW VARIABILITY AND THE CHARACTERISTICS OF INTRA-ANNUAL FLOW DISTRIBUTION

4.3.1 Introduction

The mean flow is used to describe the average amount of water that can be expected from a river or stream over any particular period of time. Because of the randomness of the input (precipitation), the flow tends to vary over time, but with some persistence in the smaller time frames (a month or less). It is important to know the extent of the variation if water management projects are to be economically and reliably carried out. The mean annual flow provides a measure of the potential supply, but the intra-annual and multi-annual variation provides information which can be used to determine the probability of experiencing deficiencies under natural conditions and the amount of storage required to reduce the probable deficiencies. In the event that there are either no or insufficient flow records near the desired location, regional techniques (see Sec. 4.1) should be applied.

4.3.2 Flow Variability

Variability in general can be characterized by three groups of descriptors, namely (a) statistics based on order-statistics, (b) statistics based on the absolute difference among the sample values, and (c) those related to statistical moments. Those descriptors based on the first two are not commonly used in hydrology.

4.3.2.1 Annual Flow Variability

The statistic most commonly used to describe variability of river runoff is the standard deviation, which is the positive square root of the variance, and the coefficient of variation. The standard deviation has the same dimension as the flow, so for comparing variabilities in streams with different size catchment areas, the dimensionless coefficient of variation should be used. An alternative is to compare the standard deviation of the logarithms of the values.

The accuracy of the variance, standard deviation and the coefficient of variation from n years of record of streamflow can be computed by procedures described in Yevjevich (1972, Chapter 9) and Rozhdestvensky (1977, Chapter 4). The accuracy of the selected parameters and of the probability distribution quantiles depends on the serial correlation (correlation between adjacent years) of the annual flows.

4.3.2.2 Intra-Annual Flow Variability

Intra-annual flow may be taken as flow in a particular season or period of the year, e.g., summer months, month or day. The monthly flow is generally used for a representative expression of the intra-annual flow. The variability of mean monthly flow is computed in the same way as for the year. In the case of groups of months, there will be as many estimates as the year is divided into groups. The variability of daily flow is not a characteristic of streamflow which usually has much application, although an index of variability has been used in regional studies (Chow, 1964, page 14-43). The distribution of daily flows (discussed in Section 4.3.4) is more usable in assessing the potential of a river to meet a specified demand.

4.3.3 Fitting Probability Distributions to Flow Data

The fitting of a theoretical distribution to flow data enables judgments to be made about the probability with which a given runoff amount will be equalled or exceeded. It has important implications in the economic planning of projects. The selection of the proper theoretical probability distribution is based on experience (Brunet-Moret, 1969). Markovic (1965) has shown that the Gaussian, log normal and gamma distributions fit well annual runoff distributions from 446 river gaging stations in the U.S. Kritsky and Menkel (Blokhinov, 1974, Chapter 1; and Rozhdestvensky, 1974, Chapter 2) have successfully applied the three-parameter gamma and Pearson Type III distributions to annual flows.

Though the Gaussian distribution fits many data, there are theoretical difficulties with its use since it assumes that runoff could be less than zero. Therefore, the distributions selected must model the asymmetry usually noted in runoff data. From the practical point of view then, it is better to use the log normal and the gamma types.
There are a number of methods by which the sample distribution can be fitted. There are graphical fitting, method of moments, method of least squares, the quantile method, the sextile method, maximum likelihood method, and plotting by the frequency factor method (Rozhdestvensky, 1974, pages 35-51, 264-277; and NERC, 1975, Chapter 1).

4.3.3.1 Graphical Fitting

The graphical fitting method is subjective and consists of fitting a curve by eye to the observed runoff data plotted on probability paper according to any of the usual plotting positions. The curve fitted this way is purely empirical and follows no known probability distribution. Extrapolation at the extreme probabilities can lead to serious errors of prediction; however, the effect of apparent high and low outliers can be discounted in construction of the curve. The graphical fitting method is a good way to test the postulation that the data are distributed according to one of the known probability distribution functions. In this case a probability paper is used where the abscissa is a linearized scale of the selected theoretical distribution. If the plotted points follow a straight line, or very nearly so, then the observed distribution is assumed to follow the selected theoretical distribution. The procedures for applying graphical fitting may be found in Yevjevich (1972, Chapter 5), Andrejanov (1975, page 14), Klemes (1973, page 16), Chow (1964, Section 8-1), Sokolov (1976, Chapter 2), Rozhdestvensky (1974, Chapter 3) and Heras (1972).

4.3.3.2 Method of Moments

In this case a decision is taken to try a particular distribution. The parameters of the distribution are estimated from the sample distribution of the observed series. The mean and standard deviation are required for two parameter distributions and the coefficient of skewness is also required for some three-parameter distributions. The estimated parameters are then used with the selected distribution model to compute the probabilities of equalling or exceeding particular runoff values. Klemes (1973, page 15-20), Chow (1964, Section 8-1), Sokolov (1976, Chapter 2), and Blokhinov (1974, Chapter 2) discuss fitting various distributions by the method of moments. Estimation of the random and systematic errors of the parameters and distribution quantities for the three-parameter gamma and the Pearson Type III distribution is described in Rozhdestvensky (1977, Chapters 3 and 4) and Rozhdestvensky (1974).

4.3.3.3 Method of Maximum Likelihood

The estimation of parameters of a selected distribution function to be used to fit an empirical distribution can also be done by the method of maximum likelihood. This is considered to be the most reliable method. The use of this method, though it gives the more efficient estimates of the parameters, is rather laborious to apply (Chow, 1964, page 8-31; Blokhinov, 1974, Chapters 3 and 4; Rozhdestvensky, 1974; pages 265-27; Gidrometeoizdat, 1973, pages 9-12; Gidrometeoizdat, 1972; pages 4-5).

4.3.3.4 Other Methods

The method of least squares can also be used to fit a theoretical distribution and involves an iterative procedure for deriving the usual parameters (Klemes, 1973, page 16; and Chow, 1964, page 8-31).

The quantile method, developed by G. A. Alekseeyev, combines the graphical and analytical methods of curve fitting. Klemes (1973, page 16), Sokolov (1976, pages 59-62), Gidrometeoizdat (1972, Chapter 2) and Gidrometeoizdat (1973, Chapter 2) describe this method.

The method of sextiles can be used to estimate the parameters in the Fisher-Tippet distribution (WMO, 1969, Chapter 5). A method of mean exceedance values has been applied by Van der Made (1969, Volume 1, pages 152-164).

4.3.4 Application of Flow Data Distributions

An analysis of annual values does not necessarily disclose water management problems because a major portion of the runoff may occur in one season. The basic measures of intra-annual flow distribution are the mean monthly and mean daily flow distributions. These distributions are used to assess the seasonal variation of flow probabilities within the year.
4.3.4.1 Monthly Flow Distribution

The techniques described for annual flow distribution are applicable to the monthly flows. In this case, it is possible that the monthly sample distributions will be fitted by different theoretical probability curves.

After obtaining the fitted distribution for each month of the year, they can be used to plot a graph showing the monthly runoff values equalled or exceeded at particular probabilities. The points of equal probabilities are then joined together. With this graph, it is possible to compare the flow distribution in any particular year with the mean monthly distribution (Klemes, 1973, pages 27-28).

4.3.4.2 Daily Flow Distribution

The distribution of daily flows is shown by the duration curve, which is obtained by arranging the mean daily flows over the period of record in descending order of magnitude. A distribution can be plotted by dividing the range between the maximum and minimum into suitable intervals, and finding the frequency of occurrences in the various intervals (Klemes, 1973, page 29; Chow, 1964, pages 14-42 to 14-44).

The primary use for this curve is in assessing the natural flow capability for hydropower plants, navigation, water pollution and water supplies. Note that all chronology and seasonal variation is lost unless seasonal duration curves are computed.

4.3.5 Absence of Observed Data

Appropriate transfer techniques will need to be applied to estimate the intra-annual and annual flow characteristics at ungaged or inadequately gaged sites. Regional analysis results may be available for the area in question or the techniques described in subchapter 4.1 may need to be applied. Some specific techniques for intra-annual flows are described in Gidrometeoizdat (1972, Chapter 3), Gidrometeoizdat (1973, Chapter 3), and Klemes (1973, page 38).


*Major English language reference.

(1) Not for sale, only special requests.
(2) Out of print, new edition foreseen in 1982 (the cost will be approximately $100).
(3) Out of print Oct 1980 (may be reprinted), see libraries.
(4) Available at 40 British pounds plus postage.
4.4 DETERMINATION OF EXTREME-DISCHARGE AND STAGE FREQUENCY CURVES FOR NATURAL AND REGULATED CONDITIONS

4.4.1 Introduction

This subchapter addresses techniques for determining extreme-discharge frequency curves for natural and regulated conditions and for both flood flows and low flows. Topics include data series, discharge-frequency curves for natural conditions, discharge-frequency curves for regulated conditions and determination of reservoir-level frequency curves.

4.4.2 Data Series

Two types of data series used for frequency analysis are the annual series and the partial-duration series. The annual series consists of one value (the highest or lowest event) that has a particular attribute for each year of record (usually water year). The attribute may be the maximum instantaneous peak flow during the water year, or the maximum daily flow during each January, or the lowest daily flow during the summer season, etc. The partial-duration series consists of all independent events either above or below a selected base. The procedure for collecting data for each type of series is described in WMO (1974, pages 5.18-5.20). For low-flow series, the streamflow record should be divided into hydrological years that start during a period of usual high flow so that the yearly low-flow periods are not likely to be divided (WMO, 1974, page 5.65).

4.4.3 Homogeneous Events

The assumption necessary for the frequency analysis of extreme events is that the events are from a representative time sample of random events. At some locations, the flood peaks are caused by snowmelt, rainstorms, or by a combination of both and at other locations the peaks may be caused by either general cyclonic storms or intense tropical storms. Therefore, when it can be shown that there are two or more distinct and generally independent causes for the extreme events, the analysis may be more reliable if the events are segregated by cause, analyzed separately, and the results combined. The procedure for combining the results is described in Sokolov (1976, pages 31-32) and Rozhdestvensky (1974, pages 42-100).

Nonhomogeneity also results from human activities and natural causes such as forest fires, earthquakes, etc. Quantitative statistical procedures should be used to test the homogeneity of the observed extreme series (Sokolov, 1976, pages 77-84; Rozhdestvensky, 1974, pages 184-225). Every effort should be made to adjust the data to obtain homogeneous series.

4.4.4 Discharge-Frequency Curves

There are two procedures that can be used to derive frequency curves - graphical and analytical. The graphical approach can be used to fit virtually any array of data, whereas the analytical approach involves fitting a preselected statistical distribution to the data. The analytically derived curve should always be compared with the plotted data to insure that the selected theoretical distribution is appropriate for the data. The analytical procedure is usually not appropriate for regulated conditions as the discharges will generally not fit any of the usual theoretical frequency distributions.

The graphical procedure involves arranging the data in order of magnitude, assigning plotting positions, and then plotting on appropriate probability paper. The details of this procedure may be found in Chow (1964, pages 8-28 to 8-29), HEC (1975, pages 4-01 to 4-07), Kite (1977, Chapter 2), Sokolov (1976, pages 28-31), and Rozhdestvensky (1974, pages 55-153).

Commonly used statistical distributions for the analytical procedure are the extremal (Type I is commonly termed Gumbel), logarithmic (log-normal) Pearson III, Log-Pearson III, exponential, and sometimes the Goodrich and normal (Gaussian) distributions. The distributions are discussed in Chow (1964), Kite (1977), WMO (1974), Sokolov (1976), IAHS (1967), Rozhdestvensky (1974), NERC (1975) and Heras (1976). See also Sec. 4.3.2.

Procedures for developing frequency curves for ungaged sites vary from region to region, therefore, local water resource agencies should be contacted in the region of interest. Some general procedures for regional analyses are discussed in subchapter 4.1 and in Chow (1964), HEC (1972), HEC (1975), Kite (1977), Sokolov (1976), Klemes (1973), and Rozhdestvensky (1974).
4.4.5. Peak-Discharge Frequency Curves for Natural Conditions

Several counties have established standardized procedures for peak-discharge frequency analysis. The United States Water Resources Council published guidelines in 1976 involving the application of the log-Pearson III distribution (U.S. Water Resources Council, 1977). These procedures have been incorporated into computer programs (see USGS, 1976 and HEC, 1976b). Programs which apply the normal, log-normal, Gumbel, and Pearson III distributions are also available.

The National Environment Research Council of Great Britain has applied several distributions to flood data in that nation (NERC, 1975). The publication describes detailed procedures on data processing, frequency methods, comparison of results and guidelines for applying standardized procedures for rainfall and snowmelt analysis. Methods for statistical analyses of maximum discharges in the USSR are described in "Instructions and Guide for Determining Design Hydrological Characteristics" (Gidrometeoizdat, 1972; Gidrometeoizdat, 1973; and Sokolov, 1966, pages 3-54).

When using statistical distributions that are described by higher statistical moments, such as coefficients of skew and kurtosis, some regionalized or weighted value should be used because the values based on the sample sizes usually available in hydrometeorologic studies have a large variability.

4.4.6. Peak-Discharge Frequency Curves for Regulated Conditions

Frequency curves for locations where flows have been altered significantly by man's activities should usually be developed by the graphical procedure because it is generally not possible to select an appropriate theoretical frequency distribution as required by the analytical method.

The usual procedure to obtain regulated extreme discharges for proposed water resource development projects is by simulating operation of the project. Techniques for developing frequency curves for regulated conditions are discussed in Chapter 7 of HEC (1975). Use of computer programs for simulating water resource systems are discussed in HEC (1976a) and HEC (1977).

Withdrawals from the stream or adjacent groundwater basins and intra-basin transfers that usually have a minor effect on the annual yield of the stream can have a significant effect on the low-flow frequency relations.

Storage-yield-frequency curves can be computed based on low-flow information to assist in evaluating the water supply potential of an existing or proposed reservoir (Gidrometeoizdat, 1972, pages 18-20 to 18-25).

4.4.7. Low-Flow Frequency Curves for Natural and Regulated Conditions

Low-flow frequency curves seldom fit one of the theoretical probability distributions. Geologic and climatic conditions often result in low-flow frequency curves of unusual shape, therefore, the graphical procedure is usually recommended. Also, many of the events may be zero and special treatment is required if the selected theoretical distribution includes a logarithmic transformation. Various aspects of low-flow frequency analysis are discussed in Chow (1964, Section 18), WMO (1974, Section 5.3.5), Rozhdestvensky (1974), Gidrometeoizdat (1972) and Gidrometeoizdat (1973).

Basin development can dramatically affect the low-flow frequency relationships. For example, a relatively moderate diversion can be neglected when evaluating flood-flow characteristics, but would significantly reduce, or eliminate, low-flows. Therefore, one of the most important aspects of a low-flow study concerns the evaluation of past and future effects of basin development.

4.4.8. Determination of River-Stage and Reservoir-Level Frequency Curves

Reservoir-level (stage or storage) frequency curves are dependent upon rate of inflow, reservoir size and method of operation. There is usually no appropriate theoretical frequency distribution that would allow for analytical analysis; therefore graphical procedures are usually recommended. Typical applications are described in Chapter 6 of HEC (1975), and Chapter 6 of Sokolov (1976).
Maximum river-stage frequency curves are often quite nonlinear and should be fitted by graphical techniques. Care must be exercised in translating the curves up and downstream as the relationships are sensitive to channel and backwater conditions. If stage data are not available at the site, it is recommended that a peak-flow frequency curve and a stage-discharge relationship be developed for the site and then combined to yield the stage-frequency curve.

References

Order From Table 1.2


*Major English language reference.

(1) Cost $50 for two volumes.
(2) Order from IAHS-France.
(3) Not available Oct 1980 (may be reprinted), see library.
(4) Available at 40 British pounds plus postage.


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*Major English language reference.

(1) Order from Virginia office of U.S.G.S.

4.5 DESIGN FLOOD DETERMINATION

4.5.1 Introduction

A design flood is represented by a flood hydrograph or peak discharge that is used as a basis for flood control planning, design or operation decisions or evaluations. The design flood may be a flood with a specified return period, or it may be a probable maximum flood for which a return period is not assigned. This section focuses on techniques for design flood determination, which includes both frequency methods and methods that utilize synthetic design storms.

4.5.2 Policy Considerations

The degree of protection to be reflected by a design flood requires policy decisions that are based on consideration of the potential consequences of exceeding the design flood. When failure of a structure would result in loss of life or extensive property damage, a high degree of protection is required against failure under the most severe flood conditions considered reasonably possible at the project site. When the consequences of failure are less severe, the choice of level of protection is best based on economic analysis in which the total annual cost (that is, cost of the structure plus flood damage) is minimized.

Procedures for determining optimum design probabilities in the U.S.S.R., United States and France are described by Sokolov(1976 Chapter 1). Table 4.5-1 illustrates criteria used in the USSR as the basis for choosing the probability of a design flood. The category of structure is a function of the size of the structure and the downstream hazard. For example, a high dam located a short distance upstream from a densely populated or highly industrialized area would be a Category I structure. See Chapter 1 of Sokolov(1976) for a definition of the categories and for a summary of similar design criteria used in other countries. Table 4.5-2 illustrates criteria developed for Great Britain (page 6 of Institution of Civil Engineers, 1978). See also Van Der Made(1969) in which design flood criteria are based on frequency curve extrapolation and an adopted risk associated with the lifetime of the project.

<table>
<thead>
<tr>
<th>CATEGORY</th>
<th>ANNUAL EXCEEDANCE PROBABILITY OF DESIGN FLOOD (in percent)</th>
<th>PERCENT CHANCE OF EXCEEDANCE IN 100-YEAR LIFE</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>.01</td>
<td>1</td>
</tr>
<tr>
<td>II</td>
<td>.1</td>
<td>10</td>
</tr>
<tr>
<td>III</td>
<td>.5</td>
<td>39</td>
</tr>
<tr>
<td>IV</td>
<td>1</td>
<td>63</td>
</tr>
</tbody>
</table>

Criteria such as that shown in Table 4.5-1 require design return periods ranging from 100 (1 percent exceedance) to 10,000 years. Because of the extremely large uncertainty associated with frequency estimates of floods having return periods greater than 100 years, it is general practice in a number of countries to use synthetic storms derived by hydrometeorological analysis as a means for determining design floods. For example, the probable maximum storm is used when the failure of a structure would be catastrophic (as for Category I in Table 4.5-1), and is intended to represent the practical upper limit of precipitation (or precipitation plus snowmelt) that is considered reasonably possible in a basin. See WMO(1974, pages A.16-A.17) for additional comments pertaining to policy considerations for design floods.

4.5.3 Design Floods of Specified Frequency

A number of alternative procedures are in use for determining the magnitude of design floods of a specified exceedance frequency or return period. These include:

a. calculation of flood frequency curves from streamflow data,

b. use of empirical equations, and

c. calculation of flood runoff from synthetic storms of specified frequency.

Procedures for calculating discharge frequency curves from streamflow data are addressed in Subchapter 4.4. Design flood hydrographs may be determined from design peak discharges and flood volumes by using maximum recorded hydrographs as patterns. Section 5.2 of Sokolov (1976) and Chapter 3 of HEC (1975) provide illustrations of this approach.
<table>
<thead>
<tr>
<th>Category</th>
<th>Initial reservoir condition</th>
<th>Dam design flood inflow</th>
<th>Concurrent wind speed and minimum wave surcharge allowance</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Reservoirs where a breach will endanger lives in a community</td>
<td>Spilling long-term average daily inflow</td>
<td>Minimum standard if rare overtopping is tolerable</td>
<td>Not applicable</td>
</tr>
<tr>
<td>B Reservoirs where a breach (i) may endanger lives not in a community (ii) will result in extensive damage</td>
<td>Just full (i.e., no spill)</td>
<td>0.5 PMF or 10,000-year flood (take larger)</td>
<td>Flood with probability that minimizes spillway plus damage costs; inflow not to be less than minimum standard but may exceed general standard</td>
</tr>
<tr>
<td>C. Reservoirs where a breach will pose negligible risk to life and cause limited damage</td>
<td>Just full (i.e., no spill)</td>
<td>0.3 PMF or 1,000-year flood (take larger)</td>
<td>Not applicable</td>
</tr>
<tr>
<td>D. Special cases where no loss of life can be foreseen as a result of a breach and very limited additional flood damage will be caused</td>
<td>Spilling long-term average daily inflow</td>
<td>0.2 PMF or 150-year flood</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>

Notes
Where reservoir control procedure requires, and discharge capacities permit, operation at or below specified levels defined throughout the year, these may be adopted providing they are specified in the certificates or reports for the dam. Where a proportion of PMF is specified, it is intended that the PMF hydrograph should be computed and then all ordinates be multiplied by 0.5, 0.3 or 0.2 as indicated.

(Source: Page 6 of Institute of Civil Engineers, London, 1978.)
A procedure has been developed in France (ICOLD, 1973) for computing a peak-discharge frequency curve for a short-term gaging station from long-term precipitation data. The method, known as the Gradex method, is also described in Section 4.4.7 of Sokolov (1976).

For situations where streamflow data are inadequate, empirical equations are in common use in some countries. In the USSR, for example, "reduction" equations that show the variation of peak discharge per unit of drainage area with either drainage area or basin lag time are widely used for drainage areas larger than 50 square kilometers. Data from Unesco (1976) and Francou (1967) may be of value for developing envelope curves and reduction equations. Other parameters typically involved in reduction equations are basin slope, daily precipitation, soil characteristics, etc. Analogues of gaged rivers are used to obtain data for determining parameters (Sokolov, 1976 and Gidrometeoizdat, 1973). For drainage areas smaller than 50 square kilometers, the Rational method is used extensively (Sokolov, 1976 and Gidrometeoizdat, 1973). See Chapter 4 of Sokolov (1976) for a detailed discussion of empirical equations, including equations for volume of runoff and for snowmelt discharge. Rodier (1965) describes a study for West Africa in which 10-year floods were developed for small tropical catchments.

Synthetic design storms are also commonly used in a number of countries as a basis for developing specified-frequency flood hydrographs. This approach is used where runoff data are inadequate for conventional frequency analysis or when peak discharges will be affected by land use changes or project development. An assumption that is generally implicit in this approach is that the exceedance frequency of the design flood hydrograph is identical to the exceedance frequency of the synthetic design storm. This assumption is questionable in situations where soil moisture conditions and precipitation distribution characteristics greatly influence flood magnitude. Whenever feasible, it is desirable to calibrate the exceedance frequency associated with a synthetic storm loss rate combination by determining the exceedance frequency of the calculated peak discharge at gaged locations from frequency curves obtained by statistical analysis.

Synthetic design storms are based on regional precipitation frequency criteria. Such criteria are obtained by first developing depth-duration-frequency relationships for point precipitation by statistical analysis of precipitation records. See, for example, Garcez (1974). Maps are then constructed with contours of precipitation amounts corresponding to selected frequencies and durations. Such maps have been prepared by the U.S. National Weather Service and are illustrated in Section 9 of Chow (1964). Design characteristics of rainfall over the USSR are described in Smirnova (1969). See also Section 4.4.3.2 of Sokolov (1976).

Point-precipitation estimates can be converted to areal-precipitation estimates by means of general relationships developed between point and area precipitation. Such relationships are derived by performing frequency studies of average precipitation recorded at a number of locations and comparing point-precipitation amounts with areal-precipitation amounts for various specified frequencies. Relationships derived in this fashion are illustrated in Section 9 of Chow (1964) and Section 4.10 of Sokolov (1976). See also Vuillaume (1974).

Once depth-duration amounts have been obtained for a design storm, the storm amounts must be distributed in time. A symmetrical time distribution satisfying the desired depth-duration relationship can be developed, or the time pattern can be based on an analysis of recorded storm distribution patterns. An example of such an analysis is provided in Huff (1967). The conversion of storm precipitation to runoff is discussed in paragraph 4.5.5.

4.5.4 Probable Maximum Flood

The probable maximum flood is the flood that would result from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. It is used in many countries as the design flood for the spillway of a structure where loss of life or major property damage would occur in the event of project failure. The probable maximum flood is determined by first deriving a probable maximum storm and then performing an appropriate precipitation-runoff analysis.

Probable maximum precipitation is estimated on the basis of storm transposition and maximization or by the use of storm models. WMO (1973), ECAFE (1967) and WMO (1969) describe the theory and techniques used by the meteorologist in deriving probable maximum precipitation. See also HEC (1975), Section 4.10 of Sokolov (1976), Section A.5.6 of WMO (1974), and Section 9 of Chow (1964).
4.5.5 Conversion of Design Storm Precipitation to Runoff

A variety of techniques are used to convert precipitation to runoff, ranging from simple empirical formulae that employ a runoff coefficient to digital models that attempt detailed simulation of various components of the hydrologic cycle (Kuchment, 1972; Kuchment, 1980). Comprehensive discussions of various conversion procedures are provided in Chapter 4 of WMO (1969, pages 137-181) and Chapter 5 of Sokolov (1976). One of the most widely used methods is the unit hydrograph method. The following discussion pertains primarily to aspects of this method.

The unit hydrograph method requires determination of parameters that govern loss (or infiltration) rates, the precipitation-runoff transformation function (i.e., unit hydrograph) and base flow. If snowmelt is a factor, additional parameters must be determined. Because it is generally necessary to divide a basin into sub-basins to adequately characterize the runoff process and develop flood hydrographs at desired locations, criteria for routing must also be developed and utilized.

4.5.5.1 Loss Rates

Selection of loss rates can be a major consideration in deriving design floods. Loss rates can be related to ground conditions where data on precipitation and runoff are available for a number of floods. Loss rates should be commensurate with the magnitude of the design storm. For specified-frequency design storms, they should be calibrated so that calculated peak discharges agree with discharges based on streamflow-frequency curves for gaged locations, provided the streamflow records at those locations are sufficient to provide reasonably reliable frequency estimates. Loss rates associated with the probable maximum storm should be the most severe that can reasonably exist in conjunction with probable maximum precipitation. Chapter 4 of HEC (1975) provides information on selecting loss rates for use in design flood computation.

4.5.5.2 Snowmelt

In many regions of the world, snow accumulates during the winter season and melts during the spring. Of major importance for snowmelt floods is the maximum accumulation of snow that occurs before the start of the snowmelt season. Frequency studies of maximum water equivalent of the snowpack at a single location or for a basin can be executed in much the same manner as for rainfall or runoff. These can be used to determine the snowpack corresponding to any specified frequency and to estimate maximum snowmelt potential. See Chapter 3 of WMO (1969) for a discussion of methods for estimating the upper limits to snow accumulation in watersheds.

Two methods commonly employed for snowmelt computations are the degree-day and energy-budget methods. The degree-day method utilizes air temperatures and a melt coefficient whereas the energy-budget method requires additional data including dewpoint temperature, solar radiation and wind speed. The energy-budget method is much more closely attuned to the physics of the snowmelt process than the degree-day method and is the preferred technique provided that sufficient data are available to select appropriate 'design' values for meteorologic parameters. See Chapter 3 of WMO (1969), Section 4.8 of Sokolov (1976), Chapter 3 of HEC (1973), Section IX of Gray (1970), and Section 10 of Chow (1964) for a discussion of methods for computing snowmelt intensity.

Precipitation that occurs during the snowmelt season is treated as snowfall and is added to the snowpack when and where air temperatures are below 1 or 2 degrees Celsius. Otherwise, the precipitation is treated as rain that is absorbed by the snowpack until saturation and melting temperatures are reached. When snowmelt occurs, it is added to any rainfall excess that occurs during a time interval. The energy-budget method for determining snowmelt accounts for the energy contributed by the rainfall.

4.5.5.3 Direct Runoff

Runoff is computed using a fixed time interval chosen to give reasonable definition to the shape of the flood hydrograph. Generally the time interval used for runoff computations is about 1/5 to 1/3 of the time of concentration of the basin. A hyetograph of precipitation excess (total precipitation minus losses) is transformed to a direct runoff hydrograph using a unit hydrograph. Assumptions implicit in application of the unit hydrograph and techniques for deriving unit hydrographs for both gaged and ungaged basins are covered in numerous
publications. See, for example, HEC(1973); Section 14 of Chow(1964); Section 5.5.4 of Sokolov(1976); Section VIII.4 of Gray(1970) and Section 4.3 of WMO(1969). If a unit hydrograph is derived from a minor-flood hydrograph, it may be necessary to modify the unit hydrograph for use with large design storms. See Section 5.5.2 of Sokolov(1976).

4.5.5.4 Base Flow

Direct runoff as determined with a unit hydrograph represents the more or less immediate runoff response to a storm event. Base flow, which generally represents runoff from antecedent precipitation events, must be added to direct runoff to obtain total runoff. For many streams, base flow is a relatively small percentage, say less than 10%, of the total runoff from major floods. Consequently, estimation of base flow is often not a critical aspect of design flood determination. Base flow estimates are sometimes based on envelope curves of base flow amounts from past floods. Base flow is discussed in Section 14 of Chow(1964); Section 5.5.2 of Sokolov(1976) and Section VII.4 of Gray(1970). A technique that can be used to estimate base flow using mathematical modeling is given in Vancon(1977).

4.5.5.5 Routing and Combining of Streamflows

In situations where it is necessary to divide a drainage basin into subbasins, runoff computed for each subbasin is routed through downstream channels and reservoirs, and combined with other hydrographs at downstream locations of interest. The passage of a flood wave through a river is described mathematically with the basic equations of unsteady flow, the St. Venant equations. Routing of the flood wave can be accomplished by a number of techniques which can be broadly categorized as (a) methods which solve the continuity equation but only approximate in some general sense a solution to the equation of motion, and (b) methods which provide a numerical solution to the complete form (one-dimensional) of both equations. The first group of methods is sometimes referred to as "hydrologic" routing methods and the second group as "hydraulic" routing methods.

At the present time (1980) hydrologic methods are probably used more extensively than hydraulic methods for routing floodwaves through natural channels. A great variety of hydrologic methods exist, some of which are described in Section 25 of Chow(1964) Section VIII.6.2 of Gray(1970) and Section 4.2.3.2 of Klemes(1973). Most of the hydrologic methods require evaluation of semi-empirical routing parameters which in some cases can only be determined by calibration with streamflow data. Use of a simple storage-routing method has been found to be useful for river routing, whereby the required storage-discharge data is acquired by means of steady-flow water surface profile computations for a range of discharges.

Conventional streamflow routing methods may be inadequate in some situations. In cases where flow emerges out onto a wide alluvial fan, the flow is essentially two-dimensional. In arid countries, it is common for substantial channel and flood plain losses to occur during the passage of a flood wave. Such phenomena are difficult to model. Subchapter 4.7 contains more detailed information on flood routing.

4.5.5.6 Reservoir Routing

Routing is performed to determine reservoir outflows that would result from a specified inflow hydrograph, with a specified stage at the start of the routing and specified operation rules. A spillway design flood is routed through a reservoir to determine the maximum pool elevation so that the elevation of the top of the dam can be established. The spillway design flood may be based on the probable maximum flood or a flood of lesser magnitude depending on the consequences of overtopping and policy considerations. Considerations regarding the choice of initial reservoir levels are discussed in Section 6.03 of HEC(1976) and in Bakhtairov(1961, pages 159-200).

Routing a spillway design flood through a reservoir having an uncontrolled spillway and/or outlet works can be accomplished very simply using a storage routing technique. A required relationship between reservoir storage and outflow is obtained by combining information from storage versus elevation and outflow versus elevation relationships. Storage routing techniques are described in many hydrology and hydraulics reference books; see, for example, Section 25 of Chow(1964) and Section VIII.6.3 of Gray(1970). Routing through a reservoir controlled by a gated spillway can be performed in a similar fashion, except that storage-outflow relationships are parametric, with gate-setting as the additional parameter.
Computer programs designed primarily to simulate the precipitation runoff process have received widespread usage for a number of years. Some representative single-event models from the United States are HEC-1 (HEC,1978), USGS Rainfall/Runoff Model for Peak Flow Synthesis (Dawdy,1972), and SWMM (Huber,1977). These three models are intended primarily for natural drainage basins and utilize the unit hydrograph approach for rainfall-runoff transformation. The HEC-1 model also has capabilities for modeling snowmelt. The SWMM model is designed primarily for relatively small urban basins and uses a kinematic wave approach for rainfall-runoff transformation. This model is intended for water quality as well as quantity analysis. The TR-20 model is particularly well suited for application to agricultural basins. A model developed in France and Canada, called the Spatial Discretisation Model, is described in Girard(1972). A brief description of some of the above models as well as several others is provided in Bowers, Chu (1977).

HEC(1966) describes a computer program that (a) computes a spillway rating curve for an assumed design head and (b) routes a spillway design flood to determine the maximum pool elevation and outflow. The program can accommodate either an ogee spillway crest with vertical walls or a broad-crested weir. The routing can be for a gated or uncontrolled spillway, and discharge from a conduit or sluice can be included.

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*Major English language reference.
(1) Order from U.S.G.S. in Denver.
(2) Out of print, see libraries.
Table 1.


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(2) Out of print; available in libraries.

(3) Not available Oct 1980 (may be reprinted), see libraries.
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permanent et transitoire adapte aux petits ordinateurs, dans resume des 
principal resultats scientifiques et techniques du service geologique national 

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(Serie Hydrologie, Vol XI, 3).

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Geneva.

**Major English language reference.
(1) Order from IAHS - France.
(2) A fourth edition of the WMO Guide to Hydrological Practices is to be published in two 
volumes in 1981.
4.6 DETERMINATION OF FREEBOARD REQUIREMENTS FOR WIND AND WAVE ACTION

4.6.1 Introduction

This subchapter addresses the definition, needs, criteria and references pertinent to the inclusion of freeboard in the design of levees, floodwalls and dams. This subchapter is intended to address requirements of freeboard resulting primarily from sustained winds. Detailed procedures for these types of computations are found in numerous references including IAEA (IAEA, 1980).

4.6.2 Definition of Freeboard

Freeboard is the vertical distance between the referenced water surface and top of structure (dam, levee, floodwall) and is used to insure the safety of structures against overtopping in case of the occurrence of the design flood. Specifically, the term "minimum freeboard" is of particular importance since it is the amount of freeboard when the water surface is at maximum design elevation. In the case of a dam, the reference level is the maximum water surface elevation attained during the spillway design flood. Or in the case of a river levee, the reference level would be the levee design flood profile. Factors which influence the dams and dike crest level, apart from the reference flood level are:

- Wind induced set up
- Seiches (oscillations)
- Short waves
- Subsidence

4.6.3 Purpose

The purpose of freeboard is to act as a safety measure against overtopping of a structure due to an adverse coincidence of physical forces acting to force the water level to rise above the maximum still water level. Overtopping of the non-overflow sections of the structure should be prevented if such overtopping is likely to cause failure of the embankment. Factors which can act independently or in concert to cause this relative rise in water surface above the normal still water design level are sustained high wind, long fetch, ice in spillways, and floods exceeding the spillway or levee design flood.

4.6.4 Channel and Levees

Freeboard is provided to ensure that the degree of protection will not be reduced by oversight of factors such as unanticipated future development or urbanization which would induce larger discharges from the design storm, embankment settlement due to subsidence, channel siltation, debris load, errors or changes in roughness or retardance coefficients, and wind-induced "set-up" and "run-up" on the levee embankment. Certain reaches require special consideration and added freeboard to cover increased uncertainties such as superelevation of the water surface at curves in high velocity channels, flow condition at junctions of two high velocity channels, debris collecting at bridge piers, transitions, and hydraulic jump basins. The amount of freeboard cannot be fixed by any general formulas which would cover all of the above phenomena. Minimum allowances are subject to an evaluation of the risk and consequences of levee failure, velocity and depth of flow and concentration at the point of overtopping. Typical minimums would be 0.3 meter in rectangular concrete sections to 1.5 meters for earth levees. If the design flood flow line for a channel is below ground line, 0.5 meter would normally suffice. A measure of the degree of safety provided by an adopted freeboard requirement is to determine what percentage increase in the design flood can be contained within the freeboard zone (United States Army Corps of Engineers, 1970, paragraph 12a). In very wide channels or where relatively long, straight reaches of channel are adversely oriented with respect to winds likely to be associated with flood runoff, wind generated waves can break against levees and run up the face of the levee. Freeboard requirements under such circumstances would be analyzed similarly to those routinely investigated in dam embankment freeboard design (Chow, 1964, section 21, page 71; Bureau of Reclamation, 1977, section 142; Rouse, 1950, Chapter 11; WMO, 1974, page A.32).
4.6.5 Large Lakes and Coastal Flood Barriers

Wind-generated waves are a major consideration in the freeboard design of levees adjacent to large lakes. Lakes having extensive water surfaces with favorable navigation, recreation and water supply resources often have associated population concentrations and structures which require protection from wind-generated wave action. Wave heights are dependent on factors such as wind speed, wind duration, fetch length and water depth (WMO, 1974, page A.32). Other factors affecting wave height and subsequent height of runup are dike or wall face slope and roughness of the dike surface (see Sec. 4.6.8). Analytic procedures for estimating the potential magnitude of these waves can be found in Davis (1969, Section 4, pages 17 through 22), USACERC (1977, Volume II, Chapter 7), WMO (1974, pages A32-A34.). Seiches are oscillations of a water volume in a lake, a harbor or a gulf. They are initiated by atmospheric pressure disturbance as is also the case with normal low pressure activities as cyclones and tornadoes. A periodic disturbance, with a period that corresponds with the period of vibration of the water volume itself will develop a resonance and these may lead to a relative significant vertical water movement. This phenomenon is typical of long lakes such as the Lake of Geneva in Switzerland or any of the Great Lakes chain on the border of the United States and Canada.

4.6.6 Dam Embankments

Freeboard for dams is often referenced to a specified water surface elevation such as normal pool, spillway design maximum pool or some intermediate reference level (WMO, 1974, paragraph A.8.1; Kondratiev, 1960, pages 5-10, 38-40). Consideration will normally be given to at least the following:

1. Height of wind tide (also referred to as "setup").
2. Height of waves induced by wind shear.
3. Wave run-up on sloping embankments.
4. Additional minimum freeboard for contingencies.

Final selection of freeboard allowance is based on the combined effects of the above considerations, taking into account the type of dam, height of dam, consequence of overtopping and risk to downstream inhabited areas (Bureau of Reclamation, 1977, pages 271-275).

4.6.7 Wind Induced Setup

Wind blowing over a water surface will cause a shear stress between the air and water surface that will make water flow in a direction with the wind. For reasons of continuity, the water has to return along the bottom as a result of the hydrostatic pressure gradient induced by the setup of water on the leeward land or levee embankment. Wind induced setup is the piling up of water along a shoreline due to wind action (WMO, 1974, page A32). This is a function of the unobstructed fetch length, water depth and wind speed in the direction perpendicular to the dam face. High velocity winds must be sustained for durations exceeding one to six hours. Wind speed measurements are made at land sites and must be adjusted to the lake surface environment. These adjustments range from 10 to 30 percent increase in velocities depending on fetch length (Davis, 1969, pages 4-18). Fetch length is often based on an "effective length" by weighting the unobstructed length components at 10° increments, 45° on each side of the perpendicular (U.S. Army Corps of Engineers, 1976, page 11-14; USACERC, 1977, Volume I, Chapter 3.4). Computation procedures are also contained in WMO (1974, pages A32-A34); and Dunn Christiansen (1975); Flather (1976); Timmerman (1975) and VNIIG (1977). Wind induced setup may also be determined according to recommendations contained in VNIIG (1977, page 166).

4.6.8 Wave Height

Wave height is the vertical distance between the wave crest and valley before it breaks. The spectrum of wave heights generated by wind shear of variable wind speed covers a wide range. The concept of "significant wave height" ($H_s$) is used to represent the mean height of the highest 33 percent of the waves generated by a specific wind velocity-duration-direction relationship. Approximately 13 percent of the waves in the series will exceed $H_s$ and the maximum wave in the series will be approximately 1-2/3 times the $H_s$. See Davis (1969, pages 4-20) and WMO (1974, page A33) for computational procedures.

Wave height and period should be computed with regard to the following zones of a water body:
Deep-water zone - with depth, $H$, greater than one half the deep water mean wave length, where the bottom does not influence the main wave characteristics;

Shallow-water zone - with depth, $H$, between one half of the medium water mean wave length and the critical wave height, where the bottom affects wave development and their main characteristics;

Surf-zone - with depth ranging from critical wave height to final wave height within which waves start breaking and are finally destructed;

Water-edge zone - with depth less than final wave height within which the flow of water from destructed waves washes the shore from time to time.

Computations of waves in the above zones under constant or variable wind may be conducted according to the recommendations given in Gidrometeoizdat (1977, pages 174-196).

When computing wave elements in currents, one should introduce adjustments for wave height and length according to recommendations contained in VNIIC (1977, pages 306-307).

In case of a contraction of a water body, when $B/D$ is less than 0.25 ($B$ = contraction width along the normal to its axis, $D$ = equivalent fetch length), wave height and wave length should be adjusted as recommended in Flather (1976, pages 308-309). A technique for estimating the probabilistic characteristics of waves in the case of variable water levels has been developed in the USSR (Kondratiev, 1972).

4.6.9 Wave Run-Up

This term is defined as the vertical height a wave will run up a sloping surface after it breaks near or on the embankment. This distance is directly proportional to the height of wave, velocity of wave, slope of the embankment and smoothness of slope surface, and inversely proportional to the wave height to wave length ratio. Run-up might typically range from 1 to 2-1/2 times the wave height for smoothed slope surfaces and about 50 percent of these values for rough-dumped riprap surfaces. See Davis (1969, pages 4-21) for procedures.

4.6.10 Design Water Level

The design water level is that reference water surface level (flat pool or water surface profile) to which "freeboard" is added to determine the finished dam, levee or dike design level. This design water level or "reference level" can be based on a hypothetical flood such as the "probable maximum flood" concept or on a probabilistic approach such as the elevation reached by some hypothetical flood, i.e., 100-year event to 10,000-year event.

The design water level can also be determined from the design storm concept or by proceeding from the statistical analyses of long-term (no less than 25 years) data of observations conducted during ice-free seasons with regard to wind-induced seasonal and annual water level fluctuations (VNIIC, 1977, pages 166-168).

The design water level probability adopted for the given project depends on the type and category of bank protection structures (VNIIC, 1977, page 165, table 90 (36)).

4.6.11 Design Wind Characteristics

When determining the elements of wind-induced waves and wind set-up for various categories of structures one can adopt various probabilities of design wind speed that depend, in turn, on the design water level probability (VNIIC, 1977, page 168, table 92 (37)). Computations of the probability curve of wind speed and wind fields with regard to its spatial distribution are discussed in VNIIC (1977, pages 168-174).

4.6.12 Contingencies

An additional 0.3 to 1.5 meters is normally added to account for unknown variables and errors in estimates, depending on the freeboard reference level, risk and height of embankment, and type of structure.

In some parts of the world, a greater emphasis is placed on multidimensional frequency analysis. It is obvious that wind and wave action are more important when superimposed on a high reference level than on a low or medium level. Multidimensional frequency analyses based on the frequency distributions of lake levels, wind setup, seiches and waves can be mutually correlated. Various combinations of components can cause a certain level and their combined frequency distribution can be computed, to be applied when assessing the embankment design level. However, this procedure is complicated by the difficulty in defining the interdependence or independence of the causative phenomenon.
References


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*Major English language reference.
(1) No preprints, journals are available.
(2) To be published in 1981.
4.7 DETERMINATION OF WATER SURFACE PROFILES

4.7.1 Introduction
A water surface profile is determined by calculating the water surface elevation at various locations along a river or stream. It is often necessary during planning and design of water resources projects to quantify the effects of those projects on water surface profiles, as discussed in HEC(1971, Section 2.04). No general technique can be efficiently used to calculate water surface elevations under all flow conditions. Therefore, choosing the appropriate computational technique for any particular problem is an important aspect of the determination of water surface profiles.

4.7.2 One Dimensional Steady Flow
A large number of problems involving flow in rivers, canals, and other conveyance channels (e.g., calculating design water surface elevations for levees, flood control channels, etc.) can be solved using techniques based on one-dimensional steady flow theory. Water surface profiles in prismatic channels can be determined using direct integration methods such as the Bresse Function or the Bakhmeteff Varied-Flow Function. These techniques and others are presented in Chow(1959, Chapter 10) and Henderson(1966, Chapter 5). The most general method for determining steady water surface profiles is the "Standard Step Method" also described in Chow(1959, Chapter 10); Rouse(1950, Chapter 9); Sokolov(1976, Chapter 6) and Henderson(1966, Chapter 5). HEC(1975, Chapter 6) illustrates the calculation of water surface profiles in complex cross sections using a small electronic calculator or slide rule. This technique is sufficiently repetitive and complex to be well suited for application to the digital computer. Several computer programs, such as HEC-2 (HEC,1979; HEC,1971) are available for this purpose. Although these programs utilize the same basic computational technique, they differ substantially in auxiliary capabilities such as calculation of flow profiles through bridges, automatic calibration, channel encroachment evaluation, graphic output display, and interactive use. A comparison of several of these computer programs is given in Eichert(1970). An efficient graphical technique is described in Kouwen(1977).

4.7.3 One-Dimensional Unsteady Flow
The theory of one-dimensional unsteady open channel flow has been well established for over 100 years. This theory is presented in Chow(1959, Part V); HEC(1975, Chapter 7); Mahmood(1975, Chapter 2); Henderson(1966, Chapter 8); and Kouwen(1977). Application of the complete theory to practical problems requires sufficient data base and a digital computer. Prior to the advent of the digital computer, many approximate techniques for routing flood hydrographs were developed which are still widely used. These techniques are described in Section 25-II of Chow(1964).

4.7.3.1 Hydrologic Techniques
The hydrologic routing techniques mentioned in subchapter 4.5 can be used to determine discharges at points of interest from which water surface elevations may be calculated by the Standard Step Method. This procedure is straightforward, relatively inexpensive, and commonly used. Details of the computations required are given in Chow(1964, Section 25-II). Hydrologic routing techniques are not well suited to highly dynamic events, such as hydropower surges, and should not be used in situations involving flow reversals.

4.7.3.2 Techniques Utilizing Portions of the Unsteady Flow Equations (St. Venant Eqs)
Approximate solutions to unsteady flow problems can also be obtained by simplifying the basic equations. Examples of such techniques are the "diffusion analogy" and the "kinematic wave". In using these techniques, care must be taken to avoid misapplication. Generalized computer programs are not available for applying these techniques to the determination of water surface profiles in channels; the kinematic wave approach, however, has been used successfully in the calculation of overland flow. Discussions of the computational details of these techniques are found in Mahmood(1975, Chapter 5) and Henderson(1966, Chapter 9).
4.7.3.3 Techniques Utilizing the Full Unsteady Flow Equations

The most general techniques for determination of water surface profiles for unsteady open channel flow are based upon the full equations which embody one-dimensional unsteady open channel flow theory. Generality is obtained, however, at the price of computational complexity. These techniques, therefore, require use of a high-speed digital computer. Highly dynamic events (such as a dam-break flood), reversing flows, and rainfall floods are examples of the broad class of problems amenable to solution using the full equations. Solution of the full equations yields time histories of both discharge and water surface elevation within the study reach. The techniques discussed below differ primarily in the mathematical scheme by which the equations are solved. Details of these schemes may be found in Mahmood(1975, Chapters 3 and 4); Henderson(1966, Chapter 8); Delft(1970) and Vedernikov(1977).

a. Method of Characteristics. - This method produces well-behaved solutions for both subcritical and supercritical flows and is, theoretically, of high accuracy. It has not, however, been successfully developed into a widely available computer program. It should be considered for problems involving prismatic channels where supercritical unsteady flow exists.

b. Explicit Schemes. - Generalized computer programs based on an explicit solution scheme are available and are being used extensively (HEC,1977). Because of the large number of calculations required with this method, economic constraints may limit the time period over which the behavior of the water surface profile can be simulated.

c. Implicit Schemes. - Generalized computer programs based on implicit solutions schemes have recently become operational (as described in Fread,1973; Abbott,1973; Vasiliev,1965 and Fread,1978). These schemes should be more economical than explicit schemes for long-term simulations and for long reaches (several hundred kilometers and more).

d. Finite Element Method. - At least one generalized computer program (HEC,1978) has recently become available. Programs of this kind exhibit many of the same operational characteristics as implicit schemes.

4.7.4 Two-Dimensional Steady and Unsteady Flow

Computations of water surface elevation contours and time histories for two-dimensional flow geometries have not yet been developed into generalized techniques. Though the hydrodynamic theory is well known, the behaviors of various solution strategies are still being explored by researchers. Specific problems can be analyzed (as documented in Mahmood,1975, Chapters 17 and 18) if the expense of utilizing an expert in the field is justified.
References


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*Major English language reference.

(1) Photostats only.
(2) Preprints not available, copies of journals are available at about $10 U.S. dollars.
(3) Photo copies only at $7.30 U.S. dollars.

Order From Table 1.2

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B-23

4.8 HYDROLOGICAL BASIS FOR DETERMINATION OF RESERVOIR STORAGE REQUIREMENTS

4.8.1. Introduction

The purpose of storage reservoirs is to redistribute streamflow to best meet certain economic or social purposes. The overall problem of reservoir design falls, therefore, into the domain of water resources management and is solved by techniques and methods of systems analysis. Hydrologic, economic, social and environmental effects are considered in such analyses.

The typical allocation of storage in a multipurpose reservoir includes flood control storage, conservation storage, hydroelectric power storage, sediment storage, and inactive storage. A single purpose reservoir may be considered as a special case of a multipurpose one. Although sometimes physically inseparable, distinction is usually made between the different components of reservoir storage. In each case, the determination of storage requirements is based on somewhat different hydrologic data.

There are two basic approaches to acquiring streamflow data for use in determining storage requirements. Data can either be from historical streamflow records or from a mathematical model which attempts to simulate the reservoir inflow. The approach selected usually depends on the availability of streamflow data.

Reservoir operation policies which are discussed in Subchapter 4.9 have a direct influence in the storage determination discussed in this chapter. A general discussion on the problems of storage requirements is given in Davis (1969); Heras (1976) and Eichert (1979).

4.8.2. Flood Control Storage

The determination of flood control storage for a reservoir project is based on hydrologic analyses that are governed by project formulation criteria. While project formulation criteria are beyond the scope of this report, the guiding formulation principles are generally that the project must be economically justified (based on benefit/cost analysis), the project should be formulated to the extent practical to maximize net economic benefits, and the project should not result in significantly increased flood hazards for any flood event that exceeds the design capacity (see U. S. Department of the Army (1975a and 1975b); Kritzky (1950); pages 53-58 of UN (1955); page 261 of James (1971); and page 30-11 of Davis (1969).

When hydrologic procedures are described in the professional literature, the degree of protection to be provided by a reservoir (which is selected based on the plan formulation criteria) is generally assumed to be known. In actual practice, however, several degrees of protection are studied and the alternative projects are compared with the plan formulation criteria before the size of the project, and therefore the degree of protection is established. The so-called "degree of protection" should not be necessarily based on conditions immediately below the reservoir since effects of various sized reservoirs on all downstream locations need to be considered.

Hydrologic procedures for determining flood control storage fall into four basic categories:

1. Detailed sequential routing studies (assuming unlimited flood control storage) to determine how much flood storage would be required to control historical floods (see page 3-10 of HEC (1971). A plot of flood control storage versus frequency of occurrence would indicate, for instance, the required flood control storage to control a 100-year flood (i.e., a one percent chance flood) to a nondamaging release.

2. Detailed sequential routing studies for all major historical floods, using a fixed amount of flood control storage, to determine the regulated flow (and perhaps the flood damages) throughout the basin. The flood control storage used in the routing would then reflect different degrees of protection at various downstream locations. Also, the sum of the expected annual damages could be used in selecting the optimum flood control storage, based on maximizing net benefits (see Eichert, 1976, and pages 258-267 of James, 1971).

3. Detailed sequential routings of a few selected historical and/or synthetic floods and multiples of those floods in order to calculate the expected annual damage for the assumed reservoir flood control storages (see Chapter 4 of HEC, 1976 or page 9 of HEC, 1979).

4. Design flood (or balanced flood) approach where a typical flood hydrograph (such as for a 100-year flood) is constructed. The balanced flood would have all of the volume-frequency statistics preserved, such as 100-year value of peak, 1-day, 3-day, 10-day, 30-day flow volumes (page 3-02 of HEC, 1971, and Chapter 3 of HEC, 1975a). A sequential routing is performed (see page 3-02 of HEC, 1971, page 87 of Klemes, 1973) using the design flood hydrograph as the inflow hydrograph.
While there are short-cut procedures that do not utilize detailed sequential routings, they are rarely of value because in these techniques reservoir releases cannot be influenced by flows downstream of the dam and by various constraints that are part of the operating rules, such as rate of change of reservoir release.

Where flood control reservoirs are assumed to have un gated outlets for making flood releases, the detailed sequential routings (based on downstream flooding) which are normally required (see examples—Exhibit 2 of Appendix 1 of HEC,1976, or Exhibit 2 of HEC,1979) can be greatly simplified such that the reservoir outflow is a single-valued function of reservoir storage. Routing techniques, such as the modified Puls method (see Chapter 3 of HEC,1976, or page 25-38 of Chow,1964) can be used in this case.

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Where sufficient time and funds are available for the study, both procedures 1 and 2 should be used. Where basins are being studied that are very large in relation to the aerial extent of storms typical of the area, the random centering of a limited number of historical storms may unduly bias the results of the frequency analysis. In that event, other equally likely storm centerings should be considered or perhaps a storm using a uniform rainfall excess over the basin should be used as indicated in procedure 3. Procedure 3, however, may not indicate large enough flood control storage requirements if channel capacities are low compared to reservoir storage volumes and flood control storage cannot be emptied before a new flood occurs.

When a reservoir has both flood control and conservation storage, studies should be made using seasonally varying storage. In many cases the timing of major storms and peak demands for conservation releases are such that more benefits can be obtained by varying the flood control storage within the year.

Procedure 3 is also useful when sufficient time and funds are not available to develop the basic flow data for all historical floods throughout the basin.

Procedure 4 is only useful for preliminary analysis of single reservoirs where funds are limited and the area being protected is close to the dam site.

Because of the importance of sequential flood routings in the determination of flood control storage, computer programs can be especially useful in reducing study costs and evaluating various alternatives. Where uncontrolled outlets for flood releases are considered, computer programs may be useful such as the HEC-1 (see HEC,1978) and HEC-5 programs (HEC,1979). Where gated outlets are involved, reservoir releases are dependent upon flows at downstream damage centers and computer programs such as HEC-5 must be employed.

Where many flood control reservoirs are being studied in a system, the use of computers is almost essential.

4.8.3. Conservation Storage

The determination of the reservoir storage requirements for all conservation purposes (except hydropower, which is covered in Section 4.8.4) will be discussed in this section. The following purposes are included when their demands can be expressed in terms of reservoir releases requirements: water supply, irrigation, hydropower, pollution control, low-flow augmentation, fish and wildlife and recreation (releases for whitewater sports such as canoeing and rafting). This determination is based on a combination of hydrologic procedures and project formulation criteria. The basic project formulation criteria discussed and referenced under flood control are also applicable to conservation studies in that projects should be economically justified and should be formulated to the extent practical to maximize net economic benefits while providing a reasonable guarantee of dependable water supply from the reservoir.

The question of a reasonable guarantee of a dependable supply of water is quite subjective and various procedures are utilized to attempt to define these criteria. Perhaps the most commonly applied criteria are that the reservoir must provide the firm (or stated) yield throughout the most critical drought period of streamflow record (see section 5.06 of HEC,1975b and page 4-15 of Davis,1969). Another criteria allows for shortages on a frequency basis so that project purposes such as water quality and irrigation can have, say, a 1 to 2 percent chance of shortage; while another criteria, the shortage index, allow shortages based on both duration and frequency of the shortage (see Section 5.07 of HEC,1975b). In addition to dependability and economics, other factors that should be considered are pool elevation fluctuation (for recreation interests), environmental effects and effects on fish and wildlife.

Specific steps to be used in determining the reservoir storage requirements for water supply and irrigation include (see page 4-15 of HEC,1971 and Chapter 2 of HEC,1977a):

a. Develop allowable shortage criteria for each project purpose (see page 5.18 of HEC,1975b).
b. Adopt routing time interval (monthly, weekly, daily or hourly) (see page 5.01 of HEC,1975b).

c. Determine and/or adapt reservoir inflows using historical or stochastic flows (see Chapter 4 and page 3.01 of HEC,1975b).

d. Estimate monthly evapotranspiration losses and monthly evaporation for conditions with the reservoir (see Section 3.02 of HEC,1975b; pages 39-47 of Lopatin,1965; and Sokolov,1974).

e. Determine seasonal patterns of demands and total annual requirements for each purpose (see page 3.11 of HEC,1975b).

f. Establish a tentative plan of operation considering flood control and reservoir sedimentation as well as conservation requirements.

g. Determine the reservoir conservation storage amount (storage-yield studies) which satisfies the criteria (in a), and which meets the seasonal demands determined (in e) using the inflows (from c), the loss rates (from d) and the operation plan (from f). This is usually an iterative process as in the case of sequential routing, since the desired reservoir conservation storage must be assumed before a sequential routing can be made and since the reservoir should draw down to the bottom of the conservation pool at the lowest elevation in the routing (see Chapter 6 of HEC,1975b).

h. Test and modify the solution to insure that criteria for all project purposes are met for all projects considered in the study.

Streamflows based on historical data from nearby stream gaging stations are almost always used in analyzing the reservoir performance. Where nearby stream gage records are not available, rainfall/runoff relationships, computer models or statistical methods are often used to generate the missing flow data (see page 4-15 of Davis,1969, and page 3.01 of HEC,1975b).

The use of synthetic flow sequences is a promising approach to obtain a better basis of average yield since it considers different sequences of flows than occurred under historical conditions (see Fiering,1967). Although the basic philosophy, theories and some procedural matters have been worked out, additional work needs to be done in the development of specific procedures before the approach can be considered fully operational for all conditions (see page 4-17 of Davis,1969, and Chapter 4 of HEC,1975b).

Over the past decade, an extensive body of literature has developed covering techniques for generating synthetic flow sequences. For the most part, the techniques have been based on the use of short-memory processes to approximate those of streamflow. More recently, long-memory processes have been introduced to approximate long-term persistence that is evidenced by many historical flow sequences. Excellent summaries on the use of short-memory processes for generating synthetic streamflow sequences, particularly Markovian-type processes, can be found in Fiering(1971) and Svanidze(1977).

The procedures used to determine the storage-yield relationship for a potential damsite may be divided into two general categories: (1) simplified analysis and (2) detailed sequential analysis. The simplified techniques are generally only satisfactory for preliminary or feasibility studies. Detailed methods are usually required in the planning and design phases (see Section 2.01 of HEC,1975b).

Many simplified methods have been proposed and are being used for preliminary type analyses; however, these methods often sacrifice accuracy in the interest of saving time and money. The most commonly used simplified method is the sequential mass curve analysis, often referred to as the Rippl Method (see Section 6.01 of HEC,1975b, page 4-11 of Davis,1969, and page A.10 of WMO,1974). The sequential mass curve method does not directly indicate the relative frequency of a shortage. However, by using a nonsequential method, a curve of yield versus shortage frequency can be determined (see Section 6.02 of HEC,1975b). A simplified nongraphic method, somewhat similar to the Rippl method, is the sequent peak method (Fiering,1967, pages 38-42).

The detailed sequential methods may be divided into simulation analyses and mathematical programming analyses. In simulation analysis, the most common detailed method, the physical system is simulated by performing a sequential reservoir routing (operation) study. In this type of study, one usually attempts to reproduce, with a high degree of fidelity, the temporal and spatial variation in streamflow and reservoir storage in a reservoir-river system. This is accomplished by a relatively complex type of bookkeeping which accounts for as many significant accretions and depletions as possible. In mathematical programming analysis, the objective is to develop a mathematical model which can be used to analyze the physical system without reproducing exact sequential occurrences.
The major limitation of sequential analysis is that the model is usually calibrated by use of historical data and, unless care is exercised by the user, past events may unduly bias the outcome (see Section 2.01 of HEC,1975b).

In the past, detailed sequential routings have been used almost exclusively for development of operating plans for existing and proposed reservoirs and reservoir systems. Preliminary planning studies, on the other hand, have been based almost entirely on simplified methods. However, the advent of the comprehensive basin planning concept, the growing demand for more efficient utilization of water resources, and the increasing competition for water among various project purposes indicate a need for detailed sequential routings in all planning studies. For reservoir systems, simplified methods will usually be inadequate for even preliminary analyses of individual projects because of the inability of the best simplified methods to account for a hydrologically integrated system operation. In order to provide the optimum benefit from water resources development, the planner and hydrologic engineer must work together in the planning stage to produce a plan of operation which will provide for efficient management of a reservoir and economical utilization of all available water. The use of detailed sequential routings by digital computer in hydrologic analyses for planning has been a useful tool in achieving these objectives (HEC,1979; Eichert,1976 and 1979; and Hveding,1976).

Detailed examples of sequential analysis are contained in Section 6.03 of HEC(1975b), Section 4-12 of Davis(1969), and page A.11 of WMO(1974).

Where two or more reservoirs are operated as a system to provide for conservation demands, additional system operation rules must be developed. These system operation rules cannot be developed by using the simplified techniques. Procedures for developing the system rule curves are found in Section 2.8 of HEC(1977a). Because of the interrelation between the reservoirs, sequential analysis is the most commonly accepted procedure for developing operating rules for conservation systems. Examples of such routings are found in Chapter 3 of HEC(1977a).

Because of the large volume of computations required, the use of computers for the analyses is highly desirable, although many such studies have been conducted by manual computations. Generalized computer models for performing these types of analyses are available, such as the HEC-3 model (see HEC,1974) which performs monthly-type conservation routings, and the HEC-5 model (see HEC,1979) which performs routings for flood control and conservation using routing intervals from one hour to one month.

4.8.4 Hydroelectric Power Storage

Most of the procedures used in determining reservoir storage requirements for other conservation purposes (see specific steps in Sec. 4.8.3) are also applicable to hydropower studies. In addition, there are a few additional items that must be accounted for in hydropower studies (see pages 5.15-5.18 of HEC,1975b). The major hydrologic estimate required is the amount of conservation storage required to provide specified energy demands or, conversely, the amount and timing of energy available from a specified quantity of storage. In addition, the following items must also be determined, taken into account and used (the first 3 are generally provided to the hydrologist by others):

1. Power load curve for the region to be served by the reservoir based on estimated future power demands (see page 5-09 of HEC,1971).
2. Specific energy requirements which the project must meet (generally monthly distribution) or in some cases power system demands and minimum project requirements (see page 5-10 of HEC,1971).
3. Plant factors (ratio of average plant generation, for routing period, to maximum possible generation) that should be used for the project.
4. Bottom of power pool elevation which satisfies other project purposes such as fish and wildlife, recreation and sediment reserve (see Sections 4.8.5 and 4.8.6 of this report) and which provides sufficient head for a reasonable power generation efficiency over the entire range of expected pool elevations.
5. Annual firm energy, the average annual power generation and the dependable capacity of the powerplant.

The major hydrologic difference between the routing studies for hydropower and other project purposes is that the quantity of water required in a given time period is a function of two independent variables—power demand and effective head. Furthermore, the quantity of water required is a direct function of the power demand but an inverse function of the head.

The relationship between head, flow and power generation is determined by use of the water power equation (see page 5-12 of HEC,1971 and page 24-7 of Davis,1969). In obtaining
the total outflow from the project, leakage through the dam must be accounted for as well as head losses through the penstock, turbine and draft tube. The tailwater elevation (see page 5-15 of HEC, 1971) is also very important in power studies since a one-meter loss in head may represent a major economic loss. Consequently, detailed studies of natural and altered tailwater conditions may be necessary. The large fluctuations of releases required to meet peak load demands can cause severe bank erosion and may require studies to determine storage requirements of downstream re-regulation dams and resulting effects on tailwater elevations.

Additional studies required for system power operation (see Section 5.08 of HEC, 1971) and rule curve operation (Section 2.8 of HEC, 1977a) are briefly described in HEC(1971), Section 6.04 and Fleshkov(1979).

Several computer models are available to simulate the power operation of reservoir projects. Most of the available generalized models use a fixed monthly time interval (such as the HEC-3 model, see HEC, 1974). In Norway and several other countries, planning of reservoirs is mostly done by models using weekly routings (see Hveding, 1976). Where no substantial power drawdown storage is available, flow duration techniques using daily flows are useful. One generalized computer model which utilizes flow duration techniques is the "HYDUR" model (HEC, 1980b).

Many site-specific models are available and discussed in technical literature, but are not of much use at different sites. The SSARR model, U.S. Army Corps of Engineers(1975), although developed for a specific system, has been generalized and has been widely used as a basin forecast and regulation model in planning and operation studies, and is used in conjunction with several hydropower models.

One generalized model, HEC-5, which provides the capability for short interval routing considering hydropower and flood control operation was developed by the United States Army Corps of Engineers, The Hydrologic Engineering Center (Eichert, 1979, HEC, 1979 and HEC, 1980a). The HEC-5 model has the ability to use any length of time step (from one hour to one month) and to mix time steps so that monthly or weekly studies can be performed for non-flood periods and hourly or multihourly intervals may be used for flood periods.

4.8.5. Sediment Reserve Storage

In order to determine the cumulative reservoir storage and elevations of the top of conservation (or normal operating pool), the top of the flood control pool and the top of the dam, a proper estimate must be made of the volume and distribution of sediment deposits which will occur over the life of the project. For very preliminary stages of planning, the total volume of sediment is the only estimate normally made. However, in more advanced studies, estimates must be made of how much deposition will occur in the flood control space and how much will occur in the conservation space. During the final hydrologic studies, an estimate of the distribution of the sediment in the reservoir is required which results in new elevation-area-capacity curves for the reservoir after one or two intervals (such as midpoint and end of project life) during the design life of the project (procedures are discussed in Chapter 5 of HEC, 1977b, and starting on page 4-3 of Davis, 1969).

The determination of the total volume of sediment requires that estimates be made for the (1) inflowing sediment, (2) the trap efficiency of the reservoir, and (3) the density of the sediment deposits (see page 4-6 of Davis, 1969). The estimate of sediment inflow, composed of suspended load and bed-load, offers the greatest opportunity for variations in procedures and results. Bed-load records are normally too difficult to collect and bed-load is usually a small proportion of the total load; therefore, bed-load is normally estimated on the basis of judgment and expressed as a percent of the suspended load (see page 4-3 of Davis, 1969). In steep mountain streams, however, bed-load may be a large fraction of the total load and can be calculated using formulas given in pages 190-230 of Vanoni (1975), Gidrometeoizdat(1973), and Karaushev(1977). Several methods are used in estimating suspended load as follows (see also Appendix 3 of HEC, 1977b):

1. Use of average recorded sediment load.
2. Computation by flow duration-sediment-rating curve method.
3. Estimates by unit yield of watershed.
4. Estimate of erosion.

A comprehensive discussion on where each of these methods should be used is covered on pages 4-3 to 4-8 of Davis(1969). The flow-duration-sediment curve method of analysis is perhaps the most desirable method of giving appropriate attention to the entire period of streamflow record. An example is shown on page 4-6 of Davis(1969).

Where sediment records are not available, the sediment load may be estimated by the sediment yield from a unit of area of the watershed as explained on page 4-4 of Davis(1969),
The trap efficiency of the reservoir can be estimated by techniques described in the following references (see Section 5.03 of HEC, 1977b, pages 4-4 to 4-6 of Davis, 1969, page 17-21 of Chow, 1964, page 337 of Lopatin, 1965, and Karaushev, 1977). Appendix 4 of HEC (1977b) describes a useful procedure to develop a trap efficiency curve from observed data.

The density of the sediment deposits may be determined by the procedure described in Davis (1969) (see page 4-7) or by techniques given in Lopatin (1965, pages 314-327).

The amount of sediment deposition that can be expected in the flood control pool can be estimated by procedures in Section 5.05 of HEC (1977b) based on the amount of time the reservoir remains in the flood control pool and the relative depth of that pool.

The traditional approach to analyzing the distribution of sediment deposits within the reservoir has relied on empirical methods which require a great deal of simplification from the actual physical problem. See Section 5.06 and Appendix 5 of HEC (1977b) for many of the weaknesses in the present empirical methods. A more analytical technique which is useful for deep reservoirs is the use of the computer program which is described in Appendix 6 of HEC (1977b). Another computer model, HEC-6, which is very useful in determining scour and deposition in both rivers and reservoirs, is particularly useful in estimating sediment deposition in reservoirs (see Appendix 7 of HEC, 1977b and HEC, 1977c).

4.8.6. Inactive Storage

Inactive storage may be provided in a reservoir project for one or more purposes, such as (1) a minimum recreation pool, (2) a minimum head for hydropower production, (3) a reserve for future sediment deposition, (4) a minimum pool for sustenance of fish and wildlife (see page 5.11 of HEC, 1975b), (5) a minimum pool for navigation in the reservoir, or (6) minimum pool for water quality in reservoir (see Pleshkov, 1975). As a rule, the reservoir is not drawn below the top of the inactive pool to serve any conservation purpose; however, the pool elevation may be allowed to vary seasonally.

Procedures for determining the inactive storage are not well documented except for individual project reports. One technique sometimes used for preliminary purposes is to provide an amount of storage in the inactive pool which is equal to or greater than the amount of space which is reserved for future sediment deposits; that is, the sediment profile is assumed to be flat. If the flat sediment pool assumption does not provide sufficient benefits for recreation, fish and wildlife, etc., then additional storage can be provided for those purposes. Any additional costs that would result because of the increased inactive storage should be allocated to the project purposes requiring the additional storage. During the detailed design of the project, inactive storage requirements are based on more detailed studies of sediment profiles at various future time intervals, (See Section 4.8.5).

References


*Major English language reference.


1977c. HEC-6, Scour and deposition in rivers and reservoirs, users manual. California.


*Major English language reference.

(1) Out of print, see libraries.

(2) Cost $50 for two volumes.


Major English language reference.
(1) Not available Oct. 1980 (may be reprinted), see libraries.
(2) Out of print, see libraries.
4.9 HYDROLOGIC TECHNIQUES FOR DETERMINATION OF RESERVOIR OPERATION PLANS

4.9.1 Introduction

The intent of an operation plan, or rule, is to define how much water is to be stored or released at each point in time for any state of a water resource system. Decisions are required for apportioning storage and release water among purposes, among reservoirs and among time periods. Decisions are generally based on both physical and economic objectives, and the solution process sometimes involves combinations of simulation and optimization techniques. Hydrologic inputs may be observed, simulated (e.g., rainfall-runoff), stochastic (using probability theory), or a combination of these. Subchapter 4.8 described the hydrologic basis for determining reservoir storage requirements. However, reservoir storage requirements cannot be calculated without considering the reservoir operation plan, which is the topic of this section. None of the references comprehensively cover this topic and therefore references on different aspects of the plan formulation are cited.

In a section on Operating Procedures, James(1971) presents six basic operating questions for which rules are needed. The first two relate to storing incoming flood waters:

1. Should current flood inflow be stored to reduce downstream damages or should it be released to save flood control space?
2. Should storage space be filled to provide future water supplies or should the space be saved for flood protection?

The flood control space is often defined by a seasonally varying storage allocation curve developed by an analysis of historical flood operations. Whether to store or release water in the flood control pool is dependent on available space, probability of filling, and consequences of increased releases vs. loss of reservoir storage space. Flood control operation is presented in Section 4.9.2, Single Reservoir Analysis, and in Section 4.9.7, Reservoir Release Determination During Flood Emergencies, which also considers forecasting in the release determination problem. The second question is answered by the storage allocation between flood control and conservation, in that water would only be stored up to the top of the conservation pool (James, 1971).

The next two operation questions presented in James(1971) are related to conservation purposes:

3. Should water in storage be released for present use or retained for future use?
4. How much water should be released from each reservoir in a system? The conservation rule curve is usually based on a seasonally varied storage level or levels. If there is plenty of water, releases are made for all project demands. If the pool level gets below the critical rule curve, then only essential demands are met and possibly at a reduced level. Conservation release determination is discussed in Section 4.9.2, Single Reservoir Analysis. Release choice among several reservoir will be discussed in Section 4.9.3, Reservoir Systems Analysis.

5. The fifth question is how to divide the released water among various potential uses. Typically all purposes are met while sufficient water is available. When supply is short, water should be supplied by priority which is subject to legal and political constraints. The quantity of water to be released for various users is sometimes dependent upon its quality, particularly its temperature because of the effects on fish.

6. The sixth question deals with selective withdrawal capability in reservoirs. The temperature of water released can affect some of the uses of water. As more reservoirs are constructed with multilevel outlets permitting selective withdrawal based on temperature, more projects may be operated for quality as well as quantity constraints.

The following sections present concepts for the analysis of a single reservoir and for reservoir systems. Some of the generalized models for simulation and optimization are briefly described. The last sections present various models for flood forecasting and reservoir release determination during flood emergencies. Some of the main principles and procedures for streamflow regulation are contained in Kartvelishvilli(1970) and Chokin(1977).

4.9.2 General Analysis Procedure

The general steps of analysis for an operation study outlined in this section are presented in Chapter 2 of HEC(1977). The basic steps are applicable for a single project or a system operating for one or several purposes.
1. **Identify the System.** To define the problem, one needs to identify the geographic, hydrologic and physical features of the reservoir system and the location of water demands.

2. **Determine Study Objectives.** State the objectives to be met by the operation. Are the study objectives to meet specified water demands, generate energy, locate facilities, improve the operation of an existing system, or any combination thereof?

3. **Determine Evaluation Criteria.** What basis for judgment will be used? Do you want to minimize shortages, maximize economic return, minimize reservoir drawdown, meet all requirements equally well, etc.?

4. **Assemble Required System Data.**
   - Hydrologic - Streamflow, evaporation rates, seepage and other losses.
   - Physical Components - Reservoir storage-elevation-area relations, spillway and outlet capacities, powerplant features, diversion capacities, etc.
   - System Requirements - Streamflow, water supply, energy, etc., requirements.
   - Operation Criteria - Which reservoir (or combination) will be used to meet which needs? Which demands have priority in water shortage situations? What are maximum flow and rate of change constraints? What are the present operation criteria of any existing system components, what can be changed, and what is fixed by legal requirements?

5. **Formulate a Mathematical Representation of the System.** The model may be in the form of a computation sheet, input cards for an existing simulation model, or a new simulation model to meet specific requirements.

6. **Validate Simulation Model.** Usually a historical period is used to determine whether the operation is realistic.

7. **Execute Simulations.** The case study approach would require the formulation of possible plans to be tested by the simulation.

8. **Evaluate Results.** Based on the evaluation criteria, the results will be compared against the study objectives to determine the adequacy of the system components and operation criteria.

The simulation strategy for a multiple-purpose reservoir can be performed using period-of-record flow data or by using isolated critical flow periods. For flood control analysis, the time interval is generally relatively small and isolated events are usually used. Once the storage allocation has been made, the operation for flood control usually starts assuming normal pool elevation, even if the historical pool level was lower. If isolated events are used, the simulated operation should carry through to the full evacuation of the flood control space. In some cases, a second flood could occur before the space has been evacuated and cause a more critical condition than the isolated events.

For most conservation purposes, a monthly time interval is sufficient to evaluate the reservoir operation in planning studies. With the use of computer models, long sequences of flow data can be used in operation studies. However, when numerous plans are being considered, the cost could be excessive. Shorter critical periods could be used to develop plans or screen alternatives. Then, with just a few alternatives, a longer sequence should be used to evaluate performance over a range of hydrologic conditions.

### 4.9.3 Single Reservoir Analysis

For a single reservoir, operation plans can be for a single purpose such as hydropower or they could include a number of conservation purposes and flood control protection for some downstream locations. During the planning and early design phase of a reservoir, simplified procedures like sequential and nonsequential mass curves can be used (HEC,1975). However, in the final stages of design and in operation planning, more detailed sequential routings are often required. Also, as a general rule, if increased accuracy is desired, shorter routing time intervals must be used.

Planning for the single purpose operation is the simplest task because there are no conflicts with other purposes or projects. Operation plans are usually developed based on meeting an expected demand during some critical hydrologic sequence to guarantee performance for any less severe sequence. The hydrologic sequence for conservation studies is usually the most severe historical event; however, synthetic sequences may also be used. Fiering (1971) provides a summary of approaches used for generating synthetic streamflow. One use of the series simulated by the Monte Carlo method is described in Svanidze (1964) and Reznikovsky (1969). Computer program "Monthly Streamflow Simulation" HEC-4 (HEC,1971) will analyze monthly streamflows at a number of interrelated stations, reconstitute missing streamflows, and generate a sequence of hypothetical monthly streamflows. One advantage of
using generated flow data is the opportunity for advanced planning of measures to minimize the effects of extreme sequences that could occur in the future (HEC, 1975). For flood control, historical and design floods are often developed to test the reservoir operation and spillway adequacy. Subchapter 4.5 presents techniques for determining design floods.

Developing operation plans for a multiple-purpose reservoir generally requires detailed sequential analysis to avoid conflicts among purposes. The selection of time interval for analysis depends on the variability of the supply and demand. Typically flood control and pumped storage hydropower operation would require short time intervals (one day or less) to adequately define operation plans and impacts on other purposes. The short time interval analysis is generally performed during extreme operating conditions to supplement the information gained from analysis using longer time intervals.

Priorities among purposes must be established during the analysis. In simulation, priorities are usually set for an operation run. The impact of changing priorities can be tested in repeated operations. In mathematical modeling, the priorities are generally defined by the operation process selecting the decision that will provide the maximum value. Many times some priorities are obvious. The question will be how to operate within the set priorities for all project purposes. Flood control usually has the highest priority; therefore, during flood operations, conservation requirements might be reduced to provide the best flood control operation.

Storage in the multiple-purpose reservoir is usually allocated into flood control space, conservation storage, and inactive or dead storage (Section 4.8). Also, an additional surcharge storage is provided above the flood control space to provide for storage of flood waters which the spillway is unable to pass. A major purpose of detailed sequential routings is to define storage allocation and operation rules that will provide for all purposes to the maximum extent possible.

Flood control storage in a multiple-purpose project is generally in conflict with conservation purposes in two ways. Flood control requires storage space that could store water for conservation purposes and flood control operation may require reduction of hydropower releases and benefits. Developing flood control operation plans usually requires routing historic and design floods through the reservoir to evaluate reservoir performance and impact on downstream damage centers. In operation studies, the absolute knowledge of future flows should not be used, because this knowledge does not exist in practice. A realistic limit should be placed on how far into the future one can look and on how accurate the future flows would be (see Chokin, 1977).

HEC (1976) presents basic principles of reservoir operation for flood control and describes procedures for establishing operational criteria. Topics include determination of reservoir release rates, regulation of a design flood, determination of outlet and spillway regulation, rule curves and multiple reservoir operation. U.S. Department of the Army (1959) discusses procedures used in establishing regulation criteria.

The basic flood operation goals can be listed as:

1. Do not endanger the dam.
2. Do not contribute to downstream flooding.
3. Do not unnecessarily store water in the flood control pool.
4. Evacuate flood control space as rapidly as possible.

Developing the operation plan to meet these goals is dependent on the amount of flood control storage available. U.S. Department of the Army (1959) describes three basic operation plans that can be used. Method A makes the maximum use of available storage during each flood event. Method B operates the reservoir based on controlling the project design flood and Method C combines the two methods. That is, operate based on the maximum use of space when feasible and use the more conservative fixed schedule during critical periods.

Obviously, operating for total control of the flood (Method A) would provide maximum flood control benefits. If sufficient flood control space is available, then storing flood flows until downstream discharges recede would be the best plan. However, care must be taken to ensure a series of smaller events do not create a more critical demand on the flood control space than a single event.

In cases where there is little or no flood control storage, the flood operation plan is generally based on a design flood approach (Method B). Without adequate flood control storage, the operation plan must protect the dam and still ensure the flood operation does not cause more flooding than would occur under natural conditions. Chapter 3 of HEC (1976) describes regulation of the reservoir design flood and Chapter 6 describes spillway operation planning.

For projects with partial flood protection, the compromise Method C approach would be applicable. Operation studies using historical and design floods would show how much storage could be used and still control the design flood. The initial flood operation would be
performed to control the flood until storage exceeded a specific level, at which the 
operation would be based on the assumption that the current flood is the design flood. At 
that point, the reservoir release will be based on pool level and inflows to the reservoir. 
The release schedule would be able to control the design flood and maintain flows at or below 
natural conditions.

The conservation storage in a reservoir may provide some flood protection; however, its 
primary purpose is to provide water whenever demands exceed supply and to provide head for 
hydroelectric energy. The upper and lower limits of conservation storage may vary 
seasonally. The upper boundary can be increased when less flood space is required. The 
minimum pool level may vary to provide a higher storage level during the recreation season 
and then to allow the pool to be lowered to supply other purposes. If several conservation 
purposes exist, then priorities will be required to ensure the highest priority demands are 
met. The conservation storage can be subdivided by allocating a portion of the total storage 
to a buffer zone. When water in storage gets down to the buffer zone, only the highest 
priorities would be supplied or supplies for all purposes will be reduced.

As with flood control, the storage allocated to conservation defines a portion of the 
operation. Within the conservation storage zone, water will be stored for future use and 
releases are only made to provide for project demands. Given the total storage, the 
operation plan generally is designed to meet specified purposes. If a safe yield approach 
was used to size storage, then all demands can be met during the historical critical period. 
The operation study would be performed using the historical flow sequence, expected demands, 
and evaporation data. During the most critical period, all demands are met while storage is 
drawn down to the minimum level. Examples of monthly operation studies are shown in Chapter 

HEC (1975) provides a complete discussion of technical procedures used to determine the 
relationship between reservoir storage and yield for a single reservoir. Procedures include 
simplified methods, nonsequential mass curve analysis and detailed sequential analysis. The 
development and use of rule curves is also presented. The rule curve development is based on 
power plant operation at a single reservoir during the period of maximum drawdown.

There is sometimes a reserve within the conservation storage to provide for events more 
severe than the historical records show or for other uncertainties. This buffer level can be 
the envelope curve based on the safe yield concept (HEC, 1975) or it can represent a zone in 
which operation policy changes. In projects with insufficient conservation storage or 
supply, the problem is where to set the buffer level.

The simulation approach would rank the demands by priority, set the level or levels for 
shortening lower priority demands, and then simulate the operation of the reservoir through 
some critical low flow sequence. By evaluating the different plans, an allocation of the 
conservation storage can be derived such that the highest priority demands will be met while 
lower priority demands are only met when the supply is above the buffer level.

Section 5-6 of Loucks (1976) describes a surface allocation model based on the expected 
net benefit for each use of the water. The results can be defined by reservoir rule curves 
for permissible release zones. An interactive computer program based on these concepts has 
developed been.

Section 25–III of Chow (1964), Reservoir Regulation, classifies and describes reservoir 
regulation for single purpose and multiple-purpose reservoirs. Part V of that section 
provides example operating schedules for multiple-purpose reservoir systems.

4.9.4 Reservoir System Analysis

Of the six basic operating questions originally presented, only one directly dealt with 
multiple reservoirs: How much water to release from each reservoir? The other five dealt 
with each individual reservoir. (Although the temperature and quality of water to release 
could also be a multiple reservoir problem.)

To have a multiple reservoir decision, there must be two or more reservoirs operating 
for a common purpose. Examples would be several hydropower reservoirs operating to meet a 
common energy requirement or several flood control reservoirs controlling flooding at a 
common downstream location. All reservoir systems can be characterized as combinations of 
tandem (in series) and parallel reservoirs.

For tandem reservoirs, the release to meet downstream demands is made from the 
downstream reservoir while releases are made from the upper reservoir to meet local needs and 
to keep the two reservoirs at some predetermined balance (usually same percent full). One 
exception to that would be when the upper reservoir is a hydropower project. Ideally the 
upper hydropower project should only release for power generation while the lower reservoir
reregulates the flow for downstream requirements.

When parallel reservoirs are operating for a common downstream point, any combination of releases that provides the required total release would be satisfactory to the downstream user. The question is how much to release from each? The simplest approach would keep the reservoirs the same percent full (balanced storage). However, the probability of refilling the storage should be considered. In both flood control and conservation operation, the reservoir most likely to refill should be emptied faster or more often than another reservoir less likely to refill.

As other project purposes are considered, the choice becomes even more difficult. If one of the reservoirs has recreation as a purpose, then during the recreation season releases should be approximately equal to inflows to keep the recreation pool full. If hydropower is a project purpose, the releases (ideally) should seldom be more than those required for energy requirements.

To adequately evaluate the operation of several reservoirs for multiple purposes requires fairly detailed sequential routings. A simulation with all the system demands and constraints can show how all purposes can be operated together. If there is some potential to maximize the return from the system operation, then optimization techniques may be employed.

HEC (1977) provides information on methods used in the analysis of reservoir systems for conservation purposes such as water supply, hydroelectric power, recreation and navigation. The reference presents a general procedure for conducting a system analysis study using simulation models (computer programs) along with illustrative examples.

4.9.5 Simulation Models

Simulation models for reservoir operation studies should model all essential characteristics of a reservoir system and be capable of handling a range of project purposes. For studies that are primarily concerned with conservation, it is often possible to exclude detailed flood control operation from major portions of the study, which enables the use of a longer time interval. Conservation simulation is frequently accomplished on a weekly or monthly time interval, with the possible exception of some hydropower studies which may require a daily or hourly interval for evaluation of run-of-river or pumped-storage projects. Flood control simulation usually requires a daily to hourly time interval. Hufschmidt (1966) provides information on how to organize a simulation study.

A large number of simulation models have been developed. A few of them have been generalized, adequately documented, and made usable on different computer systems.

Two such generalized simulation models, available through The Hydrologic Engineering Center, are HEC-3 (HEC, 1974) and HEC-5 (HEC, 1979). Both of these programs make release decisions among all reservoirs principally based on given constraints, demands, and maintaining balanced reservoirs.

The HEC-3 program is designed for monthly operation of a multiple reservoir system operating for water conservation purposes. The HEC-5 program provides for both conservation and flood control operation and operates with time increments from one hour up to one month and allows for varying the time step during the simulation. HEC-5 can also operate to determine maximum system yield for given storages. Both HEC-3 and HEC-5 can provide an economic analysis of the operation. The current capabilities for economic analysis in HEC-5 are limited to flood control and hydropower.

The Texas Department of Water Resources maintains and distributes a series of generalized computer programs including several applicable to reservoir operation studies (Texas Department of Water Resources, 1978). Their RESOF-I program is a monthly simulation program for quantity and quality routing for a single reservoir using the concept of firm yield. Their SIMYLD-II is a multireservoir simulation model capable of monthly conservation operation studies given system demands.

The simulation of reservoir temperature and quality may be necessary to meet quality requirements in the system. Reservoir Temperature Stratification (HEC, 1972) is a monthly reservoir temperature model for horizontal strata. The Water Quality for River Reservoir Systems (HEC, 1978) is a more complete model which is designed to simulate aquatic ecology in rivers and reservoirs. Simulation for water quality is discussed in subchapter 4.10 of this report.

There are numerous simulation models of the types described; however, the two agencies listed generally provide better documentation and support for their models than do other potential sources.
4.9.6 Optimization Models

The application of operations research for water resource studies has thus far received wide attention, primarily in the academic community (Loucks, 1976, Hall, 1970, Haimes, 1975, Kuchment, 1972, and Shiklomanov, 1979). Application is often difficult because of the complexity and size of water resource systems and the problem of identifying an appropriate and comprehensive set of operational constraints. Also, an optimization model must be adapted anew for each system, although recently a few optimization models have been made reasonably transferable and well documented (Texas Department of Water Resources, 1978). A review of various models developed for water resource system analysis can be found in a publication by Schanfelberger (1971). The models he surveyed use monthly or yearly time intervals and hence would not be applicable to most flood control systems. A frequent application of optimization is to maximize power production with other project purposes handled as constraints. The U.S. Bureau of Reclamation has developed a dynamic programming model to maximize power production given water demand constraints (Becker, 1974). Several examples of the use of dynamic programming and stochastic hydrology for the establishment of "Optimal Operations" were presented at the 1971 International Symposium of Mathematical Models in Hydrology under Topic 8, Optimal Operation of Water Resource Systems (Unesco, 1974).

Some governmental organizations maintain and provide reservoir models with optimization capabilities. The AL-IV program (Texas Department of Water Resources, 1978) is a general hydrologic optimization model capable of finding the minimum cost operating plan for a system of reservoirs, river junctions, canals, and river reaches. The physical system is represented by a link-node configuration.

A relatively simple generalized hydropower model in use in Norway optimizes the operation of several reservoirs. Its philosophy is based on an entirely economic operation within limits set by other purposes, such as recreation, timber floating, fishing, etc. Shortages of power are economically evaluated so as to minimize the economic losses for the community together with the other economic values of the power production. The model is used for both system planning and operation purposes (Hveding, 1970a, 1970b, 1976).

4.9.7 Reservoir Release Determination During Flood Emergencies

Reservoir operation plans for flood control could generally be improved if a timely forecasts of flows were available. For projects with little or no flood control storage, projected inflows to the reservoir would provide a basis for making early emergency releases to provide for additional flood control storage. The volume of the flood would be of primary importance, but the time to peak and peak discharge would also be of interest. For flood control projects operating to protect downstream damage centers, the locations would also be important. With projected downstream flows, the project releases could be determined to just fill the channel discharge capacity. Before it can be incorporated into an operation plan, forecasts must be timely and reliable enough to depend on for release determination.

The tools used for forecasting in rivers are basically the same as those used in hydrologic analysis (WMO, 1974). Most flood forecasts require rainfall and/or snowmelt as inputs and base runoff calculations either on empirical relationships or on water loss/runoff calculations. Another approach is to base a forecast primarily on the routing of the floodwave through the river system. A good overview of forecasting techniques is provided in Chapter 6 of WMO (1974) and in WMO (1975a). A Russian text, "Short-Range Forecasting of Lowland-River Runoff," (Alekhin, 1964) presents techniques for extrapolating flow information at a location, and for flow forecasts based on upstream data, as well as the unit hydrograph and water-balance methods. A second text, "Hydrological Forecasting," (Appolov, 1964) covers some of the same material, but also presents methods for long-range seasonal forecasts and forecasts involving river freezing and ice break-up. Forecasting problems are described in detail by E. G. Popov (1974). The USSR Hydrometeorological Centre uses mathematical models for predicting runoff from rainfall and spring snowmelt floods (pp. 121-140 of Kuchment, 1972).

With the advent of digital computers and their application in hydrology, there has been a trend toward the development of watershed models that provide a more detailed representation of the hydrologic cycle. There are presently a large number of deterministic, continuous simulation models, a few of which are used for forecasting. Three models used by the U.S. River Forecast Centers are SSARR (U.S. Army Corps of Engineers, 1975), the Sacramento Hydrologic Model (Burnash, 1973), and the National Weather Service Hydrologic Model (NOAA, 1972). All three were included in a study sponsored by the World Meteorological Organization testing ten hydrologic models (WMO, 1975b). Applications of the Sacramento model...
(pages 300-306) and the SSARP model (pages 317-344) are presented in IAHS(1968). Capabilities of HEC-1 are also presented (pages 258-267) in the same publication.

Most flood forecasting techniques require timely hydrometeorological information. The WMO Guide (1974) provides a complete description of instruments in Chapter 2, design of networks in Chapter 3, data collection, processing and publication of data in Chapter 4. The establishment of a hydrologic forecasting system generally requires a large capital investment. Before such investments are made, the value of forecasting should be considered. Day(1973) presents an evaluation approach on the costs and benefits of hydrological forecasts.

Reservoir operation should not be totally dependent on communications that are subject to failure or rely on analysis procedures that may not be implementable during emergencies. To insure project operation under extreme flood conditions, it is advisable to develop an emergency release schedule that only uses information available to the operator. HEC(1976) outlines steps required to develop gate regulation curves for a project.

The flood control operation of a project can generally be improved if the release determination can be based on the stage levels at downstream locations with damage potential (i.e., damage centers). If the downstream local flow and channel capacity are known, a release can be determined that will not contribute to downstream flooding and that will meet channel capacity at that damage center. As the number of reservoirs and damage centers increase and the travel time increases, the release determination becomes very complex. Computer program HEC-5 (HEC,1979) was developed to provide a means for determining maximum nondamaging flood control releases from reservoirs and to simulate system operation and resulting downstream flows and economic damages. The program uses a contingency adjustment to the future flow data to provide for uncertainty in the forecast. It can also have a limit placed on the number of periods of future flow it considers. If a computer program such as HEC-5 is to be used for making operational decisions on a real-time basis, a forecasting technique would be required to provide streamflows required as input. The operation of a simulation model with forecasted flows should always be critically reviewed before using the results to determine gate settings for reservoirs. A simulation model can readily be used for contingency planning by simulating system operation with alternative sets of forecasted flows (Eichert, 1975).

A plan for operating a flood control project based on always maintaining nondamaging flows at downstream locations may result in over-utilization of flood control storage space and create a situation where control over the project is lost. A compromise approach is to operate the project for nondamaging flows only up to a predetermined reservoir level. Above this level is a transition zone for which preestablished amounts of water are released as the flood progresses. Above the transition zone, releases are based on operating to prevent the dam from being overtopped. The operation in the transition zone is based on results of operation studies using historical floods. Operating for floods using this type of variable criteria should normally provide the most dependable benefits (pages 7-11 of U.S. Department of the Army,1959). Eichert, 1975 demonstrates the use of flood damage calculations to evaluate alternative operation criteria.

Reservoir releases in a multiple-reservoir system can be established on an individual reservoir basis. However, increased benefits can generally be realized by using a systems approach to release determination. System operation studies can be made using a generalized computer program such as HEC-5 (HEC,1979) to develop an interrelated set of operation rules to follow during a flood event. If real-time data or quantitative precipitation forecasts were available, a computer program could be used to determine reservoir releases, projected in time, that would provide maximum benefits for the system (Eichert,1975).
References


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(1) Preprints not available, Journals are available for $10 U.S. dollars.

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(1) About 50 copies available w/o charge from B-50 (Oct 1980), Xerox copies available for fee from B-32.

4.10 EFFECTS OF PROJECTS ON FLOW, EVAPORATION, SEDIMENT MOVEMENT, GROUNDWATER, AND ENVIRONMENTAL QUALITY

4.10.1 Introduction

Water projects have an impact on the temporal and spatial variation of both the quantity and quality of streamflow. Important factors that bear on the utility of a project, in addition to the obvious effects of reservoir storage on downstream flows, are evaporation, hydrochemical regime, groundwater, and sediment movement. In reservoir projects (Bureau of Reclamation, 1977) evaporation reduces the quantity and quality of available water, and sediment deposits decrease the amount of available space for storing water. Quantifying the effects of water resource projects generally requires detailed simulation studies. Such studies generally require the use of sophisticated digital computer systems to cope with the complexities of the physical system and/or the water management strategies. Evaluation of project performance should be based on a comparison of conditions with and without the project, taking into account watershed changes that are likely to occur throughout the project life (Pleshkov, 1975, and Shiklomanov, 1976). A general discussion of the effect of man's influence on hydrologic phenomena is given in Unesco (1980) and Unesco (1974). A comprehensive discussion on the effect of large hydraulic projects on the hydrological cycle and the environment in general can be found in Rodier (1974) and Shiklomanov (1979).

4.10.2 The Effect of Projects On Flow

4.10.2.1 Reservoir Project

Multipurpose reservoirs with both flood control and conservation (e.g., water supply, hydropower, low flow augmentation, etc.) storage are usually designed and operated to control a wide range of streamflows. Other single purpose reservoirs may only affect downstream flows over a smaller range of flows. The same analytical techniques may be used to quantify the effects from either. A simple hand computation technique based entirely on the continuity equation is adequate in many cases for routing flood hydrographs through reservoirs (HEC, 1976, page 3-01 and HEC, 1977a, page 11). Water supply, hydroelectric power, and other water conservation projects may be analyzed with mass balance techniques (Sokolov, 1974, page 89) using weekly or monthly time intervals. Reservoirs may increase the runoff in some instances because precipitation falling on the reservoir water surface is not subject to infiltration losses. Reservoirs may decrease runoff by increasing the evaporation and ground water percolation out of the basin.

Water demand, water availability, or economic aspects of the problem (in addition to the flow hydraulics) often influence the spatial and temporal discretization of a regional reservoir system. A multiple-purpose project planned for both flood control and conservation purposes is best analyzed with computer programs that simulate project operation (HEC, 1979, page 11).

4.10.2.2 Nonreservoir Projects

Nonreservoir projects for flood control include levees, channel improvements, flood-proofing of structures, and diversions into floodways.

Levees and channel improvements change the storage-outflow curve for the routing reach. This has the potential to affect the flood hydrograph, and such changes should be analyzed. A simple, yet effective, analytical approach may be developed by combining water surface profile calculations with the modified Puls routing method (U.S. Department of the Army, 1969, page 11, and HEC, 1975, page 6.01). Flood-proofing of structures usually does not alter the flow hydrograph unless potential storage areas are protected from the flood waters. Diversion of flood waters into an estuary or reservoir can be analyzed by the routing methods presented in Section 4.9 and Orlob (1977, Volume 2).

Nonreservoir projects for irrigation or water supply are not usually subject to rapid flow changes in time, and may be analyzed in terms of weekly, monthly or perhaps seasonal volumes (Sokolov, 1974).

The effects of maintenance dredging on navigation and flood control can be evaluated by use of the above techniques presented for nonreservoir projects. Generally, maintenance dredging for navigation has little effect on the flow hydrograph since it does not shift the stage-discharge rating curve. However, under certain conditions, the stage is lowered and runoff can be influenced.
4.10.2.3 Basinwide Systems

Water management schemes frequently involve multiple projects for storing runoff and/or changing the direction and timing of flow through a network of natural and man-made channels. The impacts of the projects on streamflow rates and the water balance in a region must be analyzed with great care.

Construction of reservoirs causes the change of streamflow volume and its time distribution. Streamflow volume variations may be determined by computation of water balance for the reservoir and the river reach, within the area in which the reservoir is located (Sokolov, 1974). In the case of a multireservoir system, the water balance is computed in a downstream order, beginning from the reservoirs located in the upper part and working downstream. The income part of the water balance accounts for the flows released from upstream reservoirs, lateral inflow in the river reach up to the dam site of the lower reservoir, industrial, domestic and operation wastes, diversion return flow, and additional resources available as the result of streamflow regulation in the reach between the reservoirs being studied.

The outgo part of water balance comprises the amounts of water discharging from the downstream dam, irretrievable water consumption, additional evaporation from the reservoir surface (difference between evaporation from water and land surface), and ground water losses out of the region in question. The water balance may be computed for different time intervals. Usually water balance is determined for a year or for individual months. Although simple hand computation techniques are just as applicable to basinwide development studies as to single projects, the numerous possible combinations of reservoir and nonreservoir projects which must be evaluated require electronic computers for effective analysis. Computer simulation is the most logical approach. Analysis always includes historical flows, but synthetic series (HEC, 1972a, page 5.01) should also be considered. Computer programs are available to evaluate the effects of multi-reservoir systems on runoff, flood control, hydropower requirements (HEC, 1976, page 12.01, and HEC, 1977a, pages 1-3).

4.10.3 The Effect of Projects on Evaporation

Evaporation is generally a major consideration only with conservation-type reservoir projects (e.g., water supply, hydroelectric power, irrigation). Most simulation models calculate losses from historical evaporation rates using the reservoir surface area simulated in the model and a rate of evaporation. Many evaporation computation techniques are based on energy budget considerations (Sokolov, 1974, page 89; Krambeck, 1974, page 137; and Chow, 1964, Section 11). Evaporation may be decreased by mechanical covers and artificial films (Chow, 1964, page 11-14). Aquatic plant growth can bring about significant evapotranspiration losses. For example, in tropical areas, the growth of water hyacinths can increase evapotranspiration by 2-3 times (Guadiana, 1976, Volume 3).

4.10.4 The Effects of Projects on Environmental Quality

Reservoirs and other river regulation projects can affect the quality of the riverine and terrestrial environment. Large-scale irrigation projects can substantially increase the humidity in previously arid environments. The disruption of the natural flow system can bring about changes in the surrounding flora and fauna. For the most part, environmental concerns center around changes in water quality. The water quality aspects will be emphasized here although other concerns such as the preservation of certain wildlife habitat and changes in the local climate are equally important in some areas.

4.10.4.1 Reservoir Projects

Typical water quality studies involve water temperature, biochemical oxygen demand, dissolved oxygen turbidity, and other chemical and biological properties. In general, reservoir projects affect all of these, and short interval analysis is required to capture the significant changes. Usually a critical period is selected, lasting at least 6 or 7 months, and the analysis is performed with computer oriented techniques (HEC, 1972c, page 6-01). The reservoir may be divided into layers, vertically, and water quality constituents are calculated for density stratified flow. In addition to inflow, reservoir geometry, initial reservoir elevation and reservoir releases, data must include quality constituents of the inflow and meteorological parameters.

Reservoirs are frequently constructed to improve the river water quality through low
flow augmentation. Wastewater discharges from municipal/industrial systems as well as nonpoint surface runoff all contribute various pollutants. A systems approach is necessary to be certain all components and their impacts are understood and controlled (Whipple-1977). Instream and reservoir water quality constraints must be considered in addition to other operations for hydropower, water supply, etc. General models are available to simulate water quality in a river-reservoir system (Cembrowicz,1978, HEC,1978, Straskraba,1973, and Kriventsov,1976). The application of these ecological models is limited due to the lack of adequate field verification.

Construction of large reservoirs may have significant effects on the local economy due to displacement of people and changes in fishing techniques, (e.g., Ackermann,1973). Modification of normal flows can have a large effect on the fisheries below the dam and changes in the ecosystem may increase the incidence of many human parasites (Detwyler,1971 and Ackermann,1973). In the industrialized countries, pollution may limit the use of some lakes and reservoirs.

4.10.4.2 Nonreservoir Projects

Nonreservoir projects also affect water quality (Karr,1978) because either the flow depth or velocity is changed. Even projects which change neither of these may introduce constituents into the flow that might not be there otherwise because of land use changes. It is important, when evaluating the impact of projects, to include land use and other changes not associated with the project in a base test analysis so future project conditions may be properly evaluated (Worthington,1977, Nedra,1978, Shiklomanov,1976, HEC,1977c, and Karaseff,1975).

4.10.5 The Effect of Projects on Sediment Movement

Sediment problems may be grouped into three general areas: (a) reservoirs, (b) inland rivers and streams, and (c) estuary and coastal. Major distinctions between reservoir and estuary problems are related to the facts that gravity is usually the primary driving force when flow enters a reservoir, whereas forces from tidal action and winds often dominate in estuaries. Also, major chemical changes occur as flow passes through estuaries. Estuary and coastal problems are not addressed in this guidebook.

4.10.5.1 Reservoir Projects

Sediment yield is a key aspect of reservoir sediment problems. Sediment yield refers to the total quantity of sediment material flowing into the reservoir. Several acceptable methods for calculating yield can be found in HEC,1977b, page 5.01, and Vanoni,1975, page 437, and Karaushev,1977. See subchapter 4.8.5 on reservoir storage requirements for sediment reserve.

Sediment yield is commonly quantified in units of tons/year. It is necessary to convert this quantity into a volume which requires estimates of the density of deposits. HEC,1977b, page 2.01, suggests typical values, but these should be considered as only the first approximation for any proposed site and should be supplemented if possible with measurements at existing projects in the vicinity of the proposed project.

After sediment yield is converted to a volume, a manual technique based on reservoir trap efficiency (HEC,1977b, page 5.05, and Vanoni,1975, page 580) can be used for estimating the volume of sediment which will deposit in the reservoir during the life of the project. Sediment deposits as a delta which starts at the upstream end and builds toward the dam. Therefore, all storage levels in a reservoir will eventually be affected. The backwater effects may also extend upstream as the sediment deposits progressively tend to reduce the gradient of the river and cause more deposition of the sediment load. HEC(1977b, page 5.01) presents a manual technique for distributing that volume within the reservoir. HEC(1977d)is a computer program for calculating a detailed distribution of the volume of deposits using cross sections spaced along the reservoir. Methods for evaluating sediment movement in run-of-river hydropower systems are covered in Syrojezhin(1974). Methods for calculating channel scour downstream of a dam are presented in HEC(1977b, page 5.32). That reference also describes the potential scour and deposition impacts of tributary inflows to a main channel. Since the natural river's flow variability will usually be reduced by the reservoir, channel scour downstream of the reservoir may not be as extensive as first thought. The apparent imbalance in sediment load relative to natural conditions is offset by the reduced carrying capacity because of the less variable flow regime (HEC,1977b). Advanced analytical techniques rely on the electronic computer (HEC,1977b and 1977d). Inadequate field data on
sediment properties limit the usefulness of many scour studies. The computer program of HEC(1977d) is applicable for degradation studies downstream from the dam as well as deposition of sediment in reservoirs.

Another source of sediment material in reservoirs is from aquatic vegetation. The dead vegetation collects in the reservoir and affects both the reservoir storage capacity and water quality. In tropical areas, water hyacinths can produce 1 metric ton of sediment per hectare per day (Guadiana, 1976).

4.10.5.2 Nonreservoir Projects

Most sediment problems associated with free-flowing streams, such as maintenance dredging for navigation, are sufficiently complex that only crude estimates were attempted before the electronic computer era. These problems are primarily concerned with the bed material load (Vanoni, 1975), and flow is often 2- and 3-dimensional. At this time, 1-dimensional analytical techniques such as in HEC, 1977d are the state-of-the-art. Therefore, problems must be simplified, and results should be interpreted by personnel who are experienced in river morphology. Several two dimensional sediment transport models have been developed for estuaries and may have some application in reservoir problems (Orlob, 1977). The use of physical models is often required. For example, local scour and deposition around hydraulic structures is a sediment problem which also often requires hydraulic model testing because turbulence causing the scour cannot be directly related to analytical sediment transport functions.

4.10.6 The Effects of Projects on Ground Water

Ground water storage is affected by many of man's activities which change the infiltration, percolation, aquifer, and outflow characteristics of a natural ground water system. A general description of ground water hydrology is found in Sokolov (1974, page 64) and HEC (1972b). A detailed discussion of the physical principles of water percolation and seepage is given in Bear (1968).

Man's impact on the natural ground water system begins with changing the natural land use (Unesco, 1980, Unesco, 1974, and Nedra, 1978). Urbanization tends to increase surface runoff and decrease ground water recharge. Artificial recharge methods have been developed to counteract this problem. Evapotranspiration can have a significant impact on the ground water storage (Sokolov, 1974, page 79).

River control projects such as reservoirs and channel modifications may increase or decrease net ground water recharge. The effects of reservoirs are discussed in Unesco (1980) and Unesco (1974). Changes in bank storage in rivers and reservoirs are important problems. Seepage through earth dams and levees is an important design consideration as given by Bureau of Reclamation (1977, page 165). The effects of reservoirs and nonreservoir projects on the ground water system can be modeled by the methods presented by HEC (1972b) and Fried (1975).

4.10.7 The Effects of Projects on Water Temperature

4.10.7.1 Reservoir Projects

Water temperature studies downstream of a reservoir should take into account the depth from which the water is withdrawn, as well as the amount of released water. Studies should consider the meteorological conditions as well to indicate the influence of and changes in the local climate. The interaction between air and water masses is mainly solved by energy-balance calculations, and by use of a number of empirically based equations and parameters (Orlob, 1977; Ackermann, 1973; IHD Norden, 1975, pp. 91-94; and Starosolszky, 1969, pp. 31-36). The influence of a reservoir on the temperature in the downstream river generally decreases with increasing distance from the reservoir.

4.10.7.2 Nonreservoir Projects

Projects in catchments where reservoirs are not included, will normally lead to only minor changes in the water temperature. If the water is diverted through tunnels over some distance, this may, however, lead to considerable changes in the water temperature in the main river (having reduced discharges) as well as in the river downstream of the tunnel. Water transferred from one catchment to another may also cause some changes in the water temperature in both catchments. In both cases, water and energy-budget methods are used in the calculations (IHD Norden, 1975, pp. 91-94 and Starosolszky, 1969, pp. 31-36).
In those parts of the world where the air temperature drops far below zero degrees for parts of the year, and lakes and rivers for some periods are more or less covered with ice, a project in the catchment will normally change the ice conditions.

4.10.8.1 Reservoir Projects

Reservoir regulations in the catchment that lead to increased and/or varying winter discharge will normally lead to the greatest changes in the ice conditions. The effects may be in or downstream of the reservoir. Changes in the reservoir level increase the risk for cracking of the ice cover on and along the shore which may lead to water bleeding and create problems for traffic to and from the ice, especially where the shores are steep and/or rugged (Starosolszky, 1969).

The rate of water withdrawal from the reservoir as well as the moving of the withdrawal area may also change the reservoir ice cover. The greatest changes will be downstream of the reservoir because of changes in natural runoff and temperature. Increased production of frazil and bottom ice will result in formation of ice-dams. This may increase the risk for ice-runs and floods in the river. Some of the problems may be solved by hydraulic structures as narrowing the river bed or constructing thresholds (Starosolszky, 1969). When water is released from greater depths, this water will have a higher temperature during the winter season. This may result in longer ice-free stretches and weaker ice on downstream lakes. Calculations usually involve water/air energy budget methods, as well as ice dynamics (Starosolszky, 1969).

Increased winter discharges will also increase the fresh water contribution in coastal areas which may result in increased ice formation in such areas. This effect may be reduced by mixing the fresh and salt water by deep water outlets, or bubble plants (Starosolszky, 1969, pp. 130-141).

4.10.8.2 Nonreservoir Projects

Nonreservoir projects will normally have a small impact on the ice regimes compared to projects with reservoirs. Channel improvements may, however, worsen the ice conditions due to velocities, as may the use of diverting tunnels and the transportation of water from one catchment to another (Starosolszky, 1969, pp. 130-141).

4.10.9 Use of Computer Programs for Water Quantity and Quality Investigations

Large-scale water resource project development has significant impact on the quantity and quality of the resulting water body when compared to the preproject conditions. Many techniques exist to aid the hydrologic engineer in analysis or quantification of the expected changes. Some of the less comprehensive procedures can be used for manual calculations but state-of-the-art techniques require the use of large digital computer systems. Procedures for performing comprehensive basin analyses are basically a combination of the procedures previously described. A comprehensive system approach, however, is important for today's complex water resource problems. Comprehensive analyses of man-made changes require evaluation of resultant impact at great distances downstream of the change in many cases.

Reservoirs and river systems, particularly large ones, are usually developed for multiple-purpose operation. They often provide benefits for hydropower, flood control, recreation, navigation, fish and wildlife, municipal and industrial water supply, irrigation, and, in some cases, water quality enhancement. Benefits associated with any one of these purposes (e.g., flood control) are frequently in direct competition with the benefits associated with other purposes (e.g., hydropower, recreation, water supply, etc.).

Nonreservoir projects include levees, channels, dredging, harbor development, estuary or wetlands development, locks and dams, interbasin water transfer, land use changes, sewage treatment plant siting, or siting other inputs or withdrawals to a waterway. This list is incomplete, but provides examples of the types of water resource projects which can affect the quantity and quality of flow in a nonreservoir water body. Several models may be required to evaluate all of the possible project purposes. Background information on modeling and simulation can be found in the appendix to Chapter 4.
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*Major English language reference.

(1) Out of print, see library.
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APPENDIX TO CHAPTER 4: METHODS OF COMPUTATION OF HYDROLOGICAL PARAMETERS FOR WATER PROJECTS

Generalized Mathematical Models

A generalized mathematical model (i.e., computer program) is intended for application to many specific sites having greatly different physical conditions. The model should require no modifications or only minor modification for most applications. Only generalized models for water resources applications which are readily available and well documented have been previously mentioned in the analysis procedures. Advantages and disadvantages which have direct impact on model selection will be discussed herein. The details of each model can be obtained from the referenced documents. All of the following models are coded in FORTRAN. No proprietary models are included in this presentation. Some of the following models are described in more detail by Bowers (1972). For additional discussion on other available models, see Orlob (1977, Appendix C). Some comparisons between models are made by Brown (1974) and Brandstetter (1976), but the comparisons emphasize capabilities specifically in urban hydrology.

The World Meteorological Organization (WMO) initiated in 1964 and completed in 1974, an international project on the intercomparison of conceptual rainfall-runoff models used in operational hydrological forecasting. The aim of the project was to proceed with an evaluation and intercomparison of operational rainfall-runoff models which use electronic computers to provide short-term forecasts of streamflow and to provide information and guidance on the use of such models in various forecasting situations, with regard to specific conditions and accuracy requirements. The project involved the testing of ten such models submitted by seven countries on six standard river catchment data sets from climatologically and geographically varied conditions in six countries. The final report on this project is given by WMO (1975). As a follow-up to this project, WMO has initiated in 1976 a second international project on intercomparison of models of snowmelt runoff applied to hydrological forecasting. Fourteen such models are being tested on seven standard river catchment data sets provided by seven countries.

In the following sections, some of the more commonly used generalized models are discussed. Many other models exist but cannot be described within the limitations of this publication (see also Bowers, 1977).

1. HEC-1

HEC-1 (HEC, 1973) is a single storm event model and is not capable of long-term continuous simulation. It models quantity only, but most ordinary flood hydrograph computations associated with precipitation and runoff on a complex, multichannel river basin can be accomplished with this program. Precipitation patterns must be related to a single hypothetical or recorded storm as there is no provision for precipitation loss rate recovery during periods of no precipitation.

The program may be used to optimize specified parameters of the precipitation-runoff or routing processes to achieve a best-fit with respect to observed hydrographs and precipitation.

In the process of modeling a watershed, the program provides several techniques with which to input and distribute the precipitation, treat the precipitation as rainfall or snowfall, compute rainfall and snowmelt losses and excesses, determine subbasin outflow hydrographs from unit graph techniques, and route hydrographs by hydrologic methods.
Graphical displays of hydrographs and hyetographs are optional. It has been used extensively in all aspects of flood control studies. It may be used for modeling reservoir systems where reservoir releases are based on fixed storage-discharge relations.

2. SSARR

SSARR (U.S. Army Corps of Engineers, 1975) is a continuous simulation model designed to determine streamflows throughout a river basin from snowmelt and rainfall. The model is comprised of three basic components:

a. A generalized watershed model for synthesizing runoff from snowmelt, rainfall, or a combination of the two.

b. A river system model for routing streamflows from upstream to downstream points through channel and lake storage. River flows may be routed as a function of multivariable relationships involving backwater effects from tides or reservoirs.

c. A reservoir regulation model whereby reservoir outflow and contents may be analyzed in accordance with predetermined or synthesized inflow. Reservoir releases must be input to the model or determined by fixed reservoir-discharge relations.

The successful application of the SSARR model is dependent upon derivations of the various parameters and relationships specific to a particular watershed or river system. Some of the relationships are general and therefore are applicable to many subbasins within a major drainage. Others can be specifically derived for a particular watershed.

The SSARR model is capable of simulating both large and small watersheds in climates where there may or may not be snowmelt contributions. The snowmelt relations are those derived for the mountainous areas of the Northwestern United States.

3. Hydrological Simulation Model

The Hydrological Simulation Program (Johanson, 1979 and Grimsrud, 1979), an evolution of the Stanford Watershed Model (Crawford, 1966), is a continuous simulation model of watershed hydrology. The model is for watershed analysis only (i.e., no automatic reservoir operation as a function of downstream flows, but can evaluate the hydrology of complex river basins). The model is a two-phase model in which the land surface and channel systems are treated separately. The land phase models interception, infiltration, overland flow, interflow, evapotranspiration, and depression storage. It also accounts for storage in various soil zones, groundwater storage, and groundwater flow. This phase accepts rainfall and other climatological data as input to produce inflow into the channel phase of the model. A moisture accounting procedure is used in which moisture movement is governed by the current moisture situation in the various areas and by constants which are intended to represent the physical properties of the watershed.

The channel phase of the model uses the output from the land phase to produce the outflow hydrograph for a watershed in basically two steps. First, a time-area type of routing model is used, in which a channel time-delay histogram simulates the time-delay to the outlet of the flows entering the channel system. This flow is then routed through a conceptual reservoir to simulate the storage characteristics of the channel phase. Thus the outflow from the land phase is processed by these two techniques to simulate the channel system effect and produce the outflow hydrograph.

4. SWMM

The U.S. Environmental Protection Agency's Stormwater Management Model (Brandstetter, 1976, and Huber, 1975 and 1977) simulates storm and combined sewerage systems. It computes the combined storm and sanitary runoff from several catchments and routes the flows through a converging branch sewer network. Flow-diversion structures can be modeled and storage can be simulated for both inline and overflow retention basins. An additional feature is a receiving water model which includes nonsteady formulations of hydrodynamics and mass transport for two-dimensional (vertically mixed) water bodies.

Both dry-weather and stormwater quality for suspended and settleable solids, biochemical and chemical oxygen demand, coliform bacteria, phosphorus, nitrogen, and oil and grease are computed for each modeled catchment and routed through the sewerage system. Mathematical formulations which simulate various combinations of overflow treatment processes for one treatment facility are included to evaluate the effectiveness of overflow treatment. The model is limited to the simulation of single runoff events.
5. **Tank Model**

M. Sugawara of the Japanese National Research Center for Disaster Prevention developed the Tank Model (Sugawara, 1974 and 1979) for the analysis of runoff in humid and non-humid regions. The humid region model assumes the soil is always wet (either from rainfall or groundwater) during the simulation period. The non-humid area model formulation considers the effect of soil moisture by reserving an appropriate volume in the uppermost tank. The non-humid watershed is separated into a number of wet and dry zones which vary in size according to the rainfall and resulting soil moisture.

The watershed runoff is represented by a series of vertical tanks in which outflow occurs both horizontally and vertically. The tanks' storage volume is a function of the volume of water held in the different parts of the soil column. The outlets from the tanks are a function of the rate of runoff from that soil stratum and the rate of infiltration to the next lower tank. The uppermost tank is representative of surface runoff, the second tank is representative of intermediate runoff, the third, fourth, or more tanks are representative of deep soil moisture and groundwater runoff. Evapotranspiration is subtracted from the uppermost tank. The subbasin cumulative outflow is input to the river channel where routing is also accomplished using tanks. The number and size of outlets from the tank determine the time lag and attenuation of the routed hydrograph.

The Tank model has been used successfully on several rivers in Japan. The computer code for the main parts of the program is given in the appendix of Sugawara, 1974.

6. **Constrained Linear System (CLS)**

In the constrained linear system (CLS) approach to rainfall-runoff modelling (Martelli, 1977) the continuous rainfall-runoff process is modelled as a set of simple contemporaneous linear systems. The stress is upon simplicity, and only the minimum number of linear systems is used to obtain some desired accuracy of fit between the observed and fitted flow outputs. By using quadratic programming and constrained linear systems it is possible to use multiple inputs and simultaneously arrive at individual estimates of the impulse response functions (instantaneous unit hydrographs) that have minimum variance and only a small bias. The CLS approach consists of determining rainfall excesses for a catchment area on the basis of the accumulated antecedent rainfall. For complex catchment areas with good data it is possible to separate the precipitation by sub-catchments, and also include upstream flows, if available, as separate inputs to be routed by a CLS-derived kernel function.

In practice, it has been found that three inputs are usually sufficient to give an adequate fit, and that any more do not greatly affect the accuracy of forecasts obtained because measurement errors are too big in most sets of rainfall-runoff data. The CLS approach has been found easy to calibrate and to yield comparatively accurate forecasts with minimal data and computer costs. The CLS was used in the development of the River Arno Mathematical Model in Italy and was successfully applied in 1979, as a routing model, for modelling the River Nile and its main tributaries (Martelli, 1977).

7. **STORM**

The Storage, Treatment, Overflow Runoff Model (STORM) (Brandstetter, 1976, and HEC, 1977) is a continuous simulation model designed to be used in metropolitan master planning studies for evaluating storage and treatment capacities required to reduce overflows. Pollutograph (pollutant mass-emission rates) loadings can also be computed for use in a receiving water assessment model.

Since STORM is intended for use in planning studies for screening alternatives, some of its analytical techniques are necessarily simplified. For example, the two procedures used to compute the quantity of runoff are the coefficient method and the United States Soil Conservation Service (SCS) method. In the coefficient method, a single land-use weighted runoff coefficient is applied to each hour of rainfall excess above depression storage to compute runoff. The runoff coefficient is a function of only the respective runoff coefficients for the pervious and impervious areas of the watershed. Antecedent conditions and rainfall intensity are not taken into account using this method.

The SCS Runoff curve number technique is considered to be more conceptually correct than the coefficient method. The SCS technique utilizes a nonlinear relationship between accumulated rainfall and accumulated runoff. Since STORM requires a continuous analysis, a procedure has been added that computes the curve number for each event based on the number of rainless hours since the previous runoff event and prior evapotranspiration and percolation. Unit hydrographs can be used to transform the surface runoff excesses into basin outflow hydrographs.
Loads and concentrations for six basic water quality parameters are computed. These are suspended and settleable solids, biochemical oxygen demand, total nitrogen, total orthophosphate, and total coliforms. Urban and nonurban areas may be described by up to 20 land uses. Other features of STORM are the capabilities to compute snowfall/snowmelt, dry-weather flow quantity and quality, and land surface erosion. STORM is also able to accept discharge hydrographs as input for computing the associated wash-off of constituents.

8. WQRRS

The Water Quality for River-Reservoir Systems (WQRRS) model (HEC,1978) has capability for ecologic evaluation of rivers or reservoirs. It is a dynamic continuous simulation model. The model consists of three separate but integrable modules. These are the reservoir module, the stream hydraulic module, and the stream quality module. Since each module is a stand-alone program, the reservoir, the streamflow routing, or the stream water quality module may be executed independently. The three computer programs may also be integrated for a complete river basin water quality analysis.

The reservoir section of the program estimates the water quality condition in deep impoundments that can be represented as one-dimensional systems in which the isotherms, or contours of any parameter, are horizontal. This approximation is generally satisfactory in lakes with long residence times. However, the approximation is less satisfactory in shallow impoundments or those that have a rapid flow-through time. Systems that have a rapid flow-through time are often fully mixed and can be treated as slowly moving streams using the stream section of the model.

The stream hydraulic section of the model includes six hydraulic calculation options. This module is capable of handling hydraulic behavior for both the "gradually varied" steady and unsteady flow regimes. Peak flows from storm water runoff or irregular hydropower releases can be represented.

In the stream quality module, the rate of transport of quality parameters can be represented and peak pollutant loads into the steady or unsteady hydraulic environment can be simulated. The stream portions of WQRRS have two automatic interface options for use with the STORM model. Grimsrud(1975)provides guidance in assessing the applicability of various water quality models for various planning purposes.

9. HEC-5

HEC-5 (HEC,1979) is intended to assist in planning studies for evaluating operation of systems of reservoirs and to assist in sizing the flood control and conservation storage requirements for each project recommended for the system. The program can be used in studies made immediately after the occurrence of a flood to evaluate preproject conditions and to show the effects of existing and/or proposed reservoirs on flows and damages in the system. The program should also be useful in selecting the proper reservoir releases throughout the system during flood emergencies in order to minimize flooding as much as possible and yet empty the system as quickly as possible while maintaining a balance of flood control storage among the reservoirs.

The above purposes are accomplished by simulating the sequential operation of a system of multipurpose reservoirs of any configuration for short interval historical or synthetic floods or for long duration nonflood periods, or for combinations of the two. Specifically, the program may be used to determine:

a. Flood control and conservation storage requirements (including hydropower) for each reservoir in the system.
b. The influence of a system of reservoirs on the spatial and temporal distribution of runoff in a basin.
c. The evaluation of operational criteria for both flood control and conservation (including hydropower) for a system of reservoirs.
d. The expected annual flood damages, systems costs, and system net benefits for flood damage reduction.
e. The system of existing and proposed reservoirs or other alternatives including nonstructural alternatives that results in the maximum net flood control benefits for the system by making simulation runs for selected alternative systems.

The model has the capability to determine reservoir releases based on downstream conditions and is not limited to fixed reservoir storage-outflow relations. Models having some similar capabilities have been developed in some other countries (United Nations,1978).
References


*Major English language reference.

(1) Out of print, copies can be requested from Minnesota University Library.
(2) Xerox copy available for approximately $31 U.S. dollars (Microfiche at $15).
(3) Not published yet in Sept. 1980; may not be published.


*Major English language reference.
(1) Xerox copy available for approximately $40 U.S. dollars.
(2) Limited stock available.
(3) Available from England.
(4) Available for approximately $520. U.S. dollars plus 5 percent postage.


5. Discharge of selected rivers of the world / Débit de certains cours d'eau du monde / Caudal de algunos ríos del mundo / Расходы воды избранных рек мира. Published by Unesco / Publié par l'Unesco.


10. Status and trends of research in hydrology. Published by Unesco / Bilan et tendances de la recherche en hydrologie. Publié par l'Unesco.


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22. Flood frequency computation. Methods compiled from world experience.

23. Guidebook on water quality surveys. (In press.)

25. World water balance and water resources of the earth.

26. Impact of urbanization and industrialization on water resources planning and management.

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28. Casebook of methods of computation of quantitative changes in the hydrological regime of river basins due to human activities.

29. Surface water and groundwater interaction.

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32. Application of results from representative and experimental basins.

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35. Sedimentation Problems in River Basins.

36. Methods of computation of low stream flow.


38. Methods of hydrological computations for water projects.