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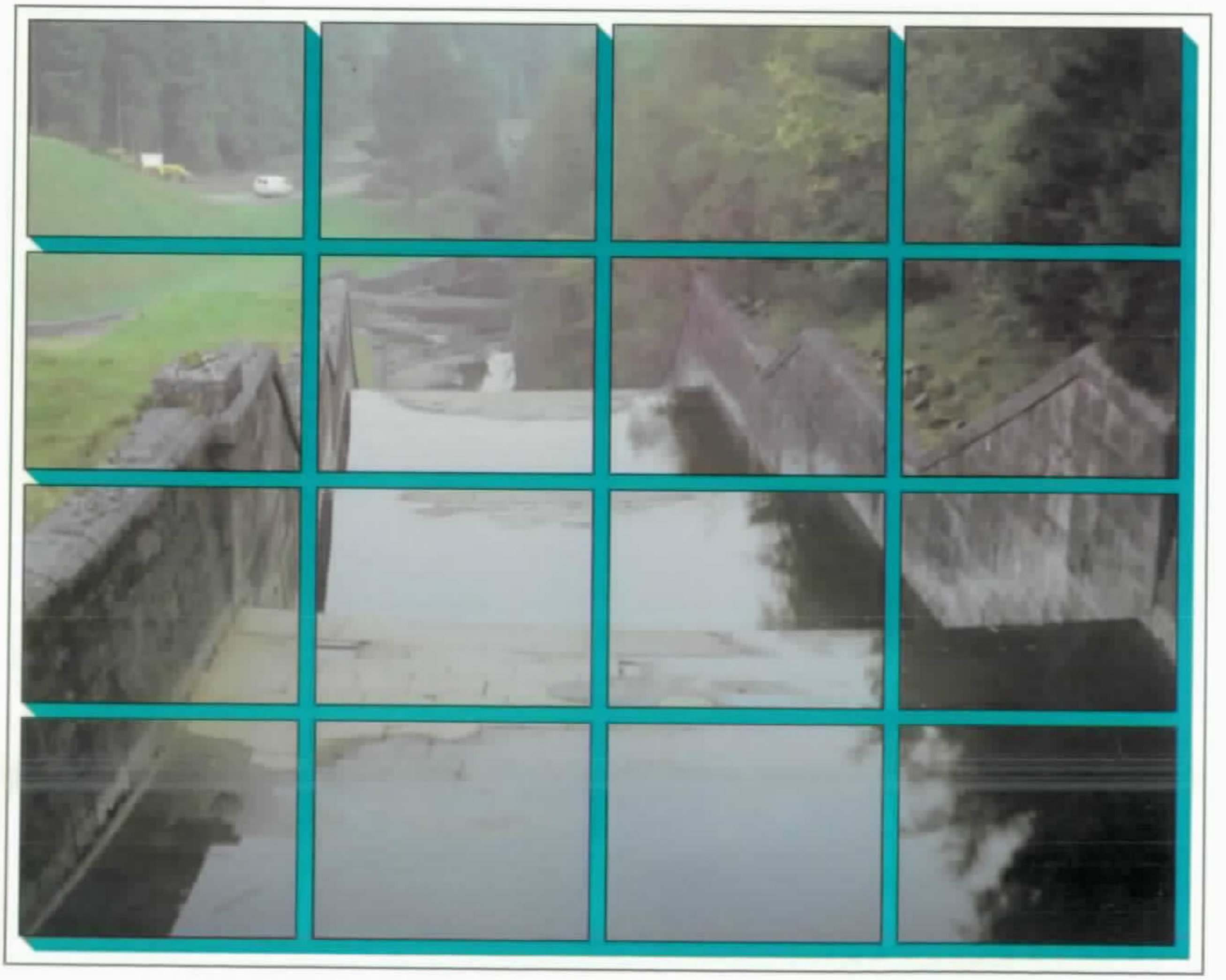

Natural Environment Research Council

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Reservoir flood estimation: another look

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Abstract

The assessment of flood risk is a vital element in the safe design, maintenance and operation of impounding reservoirs. Since the introduction of reservoir safety legislation in 1930, strengthened by the 1975 Reservoirs Act, the record of reservoir flood safety in the UK has been excellent by world standards. Nevertheless, the need remains for adequate provision to discharge floods safely at some 2400 large impounding reservoirs, many of them old and often sited above the communities which they serve.

This report summarises a study funded by the Department of the Environment's Reservoir Safety Commission under DoE Contract No. PECD7/7/135. It takes another look at reservoir flood estimation. Reference is made to developments elsewhere, but the review is primarily concerned with UK methods and experience. After identifying the procedures currently recommended, the report explores aspects of flood estimation and research which are particularly relevant to reservoir flood safety appraisals. Extensive comparisons of design floods are presented for 15 reservoired catchments.

Special topics are explored: the sensitivity of reservoir flood estimates to the precise storm duration assumed, and comparisons between 'summer' and 'winter' values of the Probable Maximum Flood (PMF) and between 0.5PMF and 10,000-year flood estimates. New algorithms are presented for 'level-pool' flood routing, with detailed examples. Controversial areas are explored, including snowmelt allowances in PMF estimation, and the incorporation of local data into reservoir design floods.

The report concludes with a selective review of procedures and developments elsewhere in the world, highlighting some of the fundamental choices in reservoir flood estimation which underlie UK practice. More than 150 references are cited, almost all of them post-dating the Flood Studies Report of 1975.

Contents

1 Introduction

1.1 Context

Flood hydrology forms an integral part of reservoir safety design and assessment. The procedures in general use in the United Kingdom were set down in the *Flood Studies Report* in 1975 and affirmed in the ICE engineering guide to *Floods and Reservoir Safety*, issued in 1978 (reprinted with minor changes in 1989).

Design standards at impounding reservoirs are necessarily high. While there have been no major flood-related incidents at UK reservoirs since July 1968, the nature of designing against the unlikely event leaves little room for complacency. An earlier study of regional flood and storm hazard (Dales & Reed, 1989) demonstrated that the clustered siting of many UK reservoirs encourages a relatively long interval between design exceedances. However, a corollary is that, when such an event occurs, there may be multiple exceedances, affecting several reservoirs in a district or region. A review of a number of aspects was therefore commissioned by the UK Department of the Environment as Project No. PECD7/7/181, the terms of reference of which are reproduced in Appendix A.

This report represents the main outcome of the study, the scope of which can best be summarised under three headings:

- a review of the existing procedures,
- investigation of specific topics, and
- a review of relevant research.

1.2 Comprehensiveness

The estimation of very rare floods, or of the Probable Maximum Flood (PMF), can raise questions that encompass the practical, the scientific and the philosophical. In a study of modest proportions (3.5 man-years in the context of more than 2000 major UK dams) it is inevitable that coverage has been uneven. While this may disappoint those at the practical or scientific extremes of the subject, it is hoped that the drawing together of information within a single report provides some compensation. To this end, particular attention has been paid to compiling a reference list that is extensive, accurate and up-to-date.

1.3 Content

The review of current procedures comprises Chapters 2 to 5 and Appendix B. This review includes an analysis of catchment characteristics (Chapter 3) and flood estimates (Chapters 4 and 5) for typical reservoired catchments, made possible by a survey of several major users of the procedures. Comparisons are made with the gauged catchments on which development of the estimation methods was based.

Investigations of specific topics follow, grouped under these headings: rainfall (Chapter 6), snowmelt (Chapter 7), rainfall-runoff modelling (Chapter 8) and reservoir routing (Chapter 9). The rainfall chapter includes an examination of the significance of heavy rainfall in historical dam safety incidents in the UK, but also considers the possible significance for reservoir flood estimation of recent research on rainfall frequency. The discussion of snowmelt is centred on a literature review and is guided towards the vexed question of a suitable allowance for snowmelt in the estimation of very rare floods.

It proved impractical within the study to make substantial headway in assessing the suitability of particular rainfall-runoff models for use in the small upland catchments typical of reservoired sites. Chapter 8 points to some related studies where useful progress has been made.

Particular emphasis was given to the production of reservoir routing software that is both reasonably general and 'user-friendly', well formulated mathematically, and accompanied by ample worked examples. Presentation of this material is divided between Chapter 9 and Appendix C.

Procedural developments in reservoir flood estimation are discussed for six countries. To have carried out a comprehensive review of research and development worldwide would have consumed all the resources available to the project. Moreover the outcome might have had no greater impact than the review presented in Chapter 10. This uses contrast rather than consensus to indicate those areas where UK practice may be ahead or behind.

The main conclusions of the study are drawn together in the summary (Chapter 11).

2 Survey of current procedures

2.1 What they are

The procedures currently recommended for reservoir flood estimation are defined jointly by four documents:

A *Floods and Reservoir Safety: An Engineering Guide* (ICE, 1978; 2nd edition, 1989)

B *Flood Studies Report* (NERC, 1975)

C *Flood Studies Supplementary Reports* (Institute of Hydrology, various dates), and

D *Methods of Flood Estimation: A Guide to the Flood Studies Report* (Sutcliffe *et al.*,1978).

The *Engineering Guide* (Document A) is the premier reference. An interim review was carried out after five years' experience (Clarke & Phillips, 1984). A second edition of Document A was published in 1989 and its preface summarises the amendments incorporated therein. It anticipates that comprehensive updating of the guide will follow the completion of a number of relevant research projects.

Reference to the *Flood Studies Report* (FSR; Document B) is written into the *Engineering Guide*. However some of the methods given in the FSR have been subject to revision or extension. Thus it is necessary for the user to refer also to the *Flood Studies Supplementary Report* (FSSR) series (Document C).

To date, there have been 18 FSSRs. These are issued to some 600 subscribers to the series. In 1981, subscription to the FSSR series was made automatic on purchasing the FSR, copies of which continue to be sold at the rate of almost one per week. Of particular relevance to reservoir flood estimation are FSSRs 10 and 16.

FSSR 10 sets out a method for calculating flood estimates for reservoirs in cascade, i.e. reservoirs which have one or more reservoirs within their drainage area. The report supersedes the specific advice given on page 34 of Document A.

FSSR 16 presents revised parameter estimation equations for the FSR rainfall-runoff method of flood estimation. This is directly relevant to reservoir flood estimation since Documents A and B advise that reservoir flood estimates should be based only on the rainfall-runoff approach.

The exception to this is that, for screening purposes, it is permissible to use a simple regression equation for the Probable Maximum Flood (PMF). This is given as an Appendix in Document D. The equation was developed by Farquharson *et al.* (1978) to provide a more faithful approximation of the full PMF method than the partly graphical method given in Appendix 1 of Document A.

It is inadvisable for users to refer to Document D other than for the PMF regression equation. Because of the extensive revisions issued in the FSSR series, it is preferable to refer only to Documents A, B and C.

The interim review of the *Engineering Guide* (Clarke & Phillips, 1984) brought the following matters to the attention of Panel Engineers:

- the Engineering Guide is not mandatory;
- the 'design flood' is one which, if exceeded, is likely to cause a breach; in most cases it would be considerably greater than the spillway design flood;
- the Panel Engineer, in consultation with the owner, is to use discretion in selection of the 'design flood';
- the 'community', for delineation of Category A dams, is one of not less than ten persons, and
- adoption of PMF for the design of a new dam might not always be appropriate.

Other reports in the FSSR series have some relevance to reservoir flood estimation. For example, FSSR 13 summarises guidance for the incorporation of local data into flood estimates, though it does not explicitly refer to reservoir applications. As discussed below, users appear to be especially reluctant to incorporate local data in flood calculations relating to reservoir safety assessment.

FSSR 18 sets out a procedure for assessing the collective risk of a design exceedance occurring at one of a network of sites which are sensitive to heavy rainfall. Examples are given of its application to groups of reservoirs.

CIRIA Technical Note 100 (Hall & Hockin, 1980) gives an alternative procedure for T-year flood estimation for the design of flood balancing ponds. Publication of a revision to TN100 is imminent. It appears proper that this separate design procedure exists, both because many balancing ponds are small structures not coming under the Reservoirs Act and because the problems faced in siting and sizing a balancing pond require special guidance. However, it is perhaps helpful to the user (who may need to consider both procedures) that the revision narrows the difference between FSR and TN100 methodology.

The recommended procedures for flood estimation are summarised in Appendix B, both for the rainfall-runoff method relevant to reservoir flood estimation and for the statistical method. In particular, attention is drawn to some regional variations in methodology that may have relevance in particular applications.

2.2 How they are used

It appears, from discussion with consulting engineers and others concerned with reservoir flood estimation problems, that the current procedures are generally followed closely in reservoir safety assessments.

Form of calculations
The FSR presents the flood estimation procedures in a rather mixed format. Chapter 6 of Volume I describes the T-year flood and PMF procedures step-by-step. Generally the hydrological calculations are defined by equations. In contrast, the storm calculations (originating in Volume II) are mainly given in tabular form. In addition, certain key steps are defined neither by equation nor by table but diagrammatically.

For example, the areal reduction factor required to convert estimates of rainfall at a point to average rainfall over a catchment is given both in tabular and in graphical form: neither is immediately suitable for computerisation. Moreover, the diagram does not define areal reduction factors for catchments having an area less than 10 km^2, which is inconvenient given that reservoired catchments in the UK are typically much smaller (median area *c.* 4 km^2).

In reservoir applications it is generally necessary to have some computational aid: otherwise the convolution of the design storm with the unit hydrograph is time-consuming. Computer software is also helpful in the subsequent 'routing' of a design hydrograph through the reservoir, to take account of the delay and attenuation effects imposed by the temporary storage of water above the overflow level of the reservoir.

Software for reservoir flood estimation
Many users of the Flood Studies Report have developed their own computerised implementations. Sometimes this was just a case of exploiting the convolution and routing programs given in FSR I.6. However, major users have also computerised the rainfall calculations. Multiple calculations may be required, particularly in complicated reservoir systems, where it is often necessary to consider a number of design storm durations and, sometimes, more than one design standard.

Difficulties in computerising the rainfall calculations were eased somewhat by the formulae presented by Keers & Wescott (1977). However, their report contains a number of damaging typographical errors. The formulae underlie the Meteorological Office's ITED software, which is used to service requests for rainfall frequency estimates.

FLOWPRED
Prior to 1988, the most widely used commercial implementation of the FSR flood estimation procedures was the FLOWPRED package (see Archer & Kelway, 1987). In its initial form, FLOWPRED used a 'batch-processing' style. However, it has been updated both to incorporate the revisions to the rainfall-runoff method presented in FSSR 16 and to offer interactive facilities typical of modern microcomputer packages.

One feature of FLOWPRED is non-standard: it offers a gridded database of key rainfall variables, thereby saving the user the labour of mapwork. However, the distance interval at which these data are gridded is too coarse to guarantee close agreement with manually-derived estimates on small catchments. We therefore suggest that users of FLOWPRED should shun the option to use the gridded database, or should at least carry out checks.

Micro-FSR
In March 1988, the Institute of Hydrology launched the Micro-FSR package. The package provides a wide range of options, allowing the user to carry out calculations under a number of assumptions: for example, comparing calculations using FSSR 16 with those obtained with the original FSR rainfall-runoff method. A particular feature of the package is that it guides the inexperienced user through the various steps and options, always highlighting the standard option that conforms to the currently recommended procedures defined by Documents A to D (see Section 2.1). A second version of the package has been completed, which enhances many aspects of the user interface and,

importantly, introduces software for reservoir routing.

In general, Micro-FSR follows the policy adopted in the FSSR series that the methods are consistent with national recommendations. This leads to some confusion in regions where analyses have been reworked and some variation in methodology introduced, to which Appendix B provides some respite.

2.3 Problems arising

Interpretation of recommendations
When procedures are subject to major updates, it is inevitable that confusion sometimes arises. One example concerns the allowance for urbanisation in the estimation of percentage runoff. In the original version of the rainfall-runoff method (FSR Vol. I, 6.8.2), the allowance was made as part of the calculation of Standard Percentage Runoff (SPR): this was estimated from the fractions of the *rural* catchment that fall into each of the five Winter Rainfall Acceptance Potential (WRAP) classifications at 1:625,000 scale, and from the fraction of the catchment mapped as an urban area at 1:50,000 scale. In the revised rainfall-runoff method (FSSR 16), SPR is taken to represent the standard response of the catchment as if it were entirely rural: thus it is estimated from the fractions of the *entire* catchment that fall into each WRAP class. The allowance for urbanisation is made in a subsequent step. Either procedure is reasonably clearly set out in the relevant reference but it is recognised that confusion can afflict those who encounter both versions.

It is not uncommon for some aspect of the unit hydrograph to be misconstrued. For example, Jefferies *et al.* (1986) mistake unit hydrograph time-to-peak (Tp) for the rainfall-runoff lag time (LAG), also used in the FSR rainfall-runoff method.

Presence of a reservoir
The very presence of a reservoir can lead to some difficulties in methodology. If the surface area of the reservoir forms an appreciable fraction of the catchment, it is clearly necessary to take account of the reservoir routing effect (which is likely to be appreciable). It is also desirable that rain falling directly onto the surface of the reservoir should not be subject to infiltration losses. This can be achieved by excluding the surface of the reservoir from the catchment on which the inflow hydrograph is synthesised: rain falling directly onto the reservoir is then included as part of the reservoir routing calculations.

If the reservoir extends well up the catchment, the standard procedure for delineating the main stream — and measuring its length and slope from 1:25,000 maps — may lead to a length that is too long and a slope that is too shallow. This may in turn lead to overestimation of the catchment response time to heavy rainfall. In such cases, the recommended guidance (FSSR 10, page 4) is to take the main stream length and channel slope to the perimeter of the reservoir (rather than to the dam site). If there is no obvious main tributary to the reservoir, the recommendation is to take the stream draining the largest area or, if that is unclear, to calculate main stream length and slope for a 'typical' tributary.

Partly to avoid this problem, and partly through a belief that a spatially more detailed synthesis of flood response is more realistic, one user applied the rainfall-runoff method on a subcatchment basis. This is generally essential where there is an upstream reservoir (see FSSR 10), which makes the argument — which is generally desirable in other cases — difficult to reject. Perhaps the one weakness of adopting a subcatchment approach is that the greater detail might be mistaken for a better model. If, as is usually the case, the parameters of the rainfall-runoff model are estimated from catchment characteristics (rather than from locally observed flood data) the estimates are likely to be of poor quality, not justifying the refinement.

Other problems stem from ambiguities and anomalies in the procedures. These are most prevalent in the estimation of the Probable Maximum Flood (PMF).

Ambiguity in probable maximum rainfall interpolation
The PMF procedure calls for interpolation to determine values of maximum rainfall (P) for durations (D) intermediate to the 2- and 24-hour durations for which maps are presented in the FSR. Volume I indicates that linear interpolation on lnP versus lnD should be used whereas Volume II indicates linear interpolation on P versus lnD. An unnumbered sheet issued in the FSSR series advises that the latter should be adopted.

Anomaly 1: "Urbanisation reduces percentage runoff in PMF case"
FSSR 16 introduced revised estimation equations for key parameters in the rainfall-runoff approach, notably for percentage runoff (PR) and unit hydrograph time-to-peak. The method by which PR is estimated includes an allowance for catchment urbanisation. The allowance (applied at all return periods) is to

assume that 70% runoff occurs from the impervious parts of the urban area (roofs, roads, etc.). The intention is that the adjustment should represent the higher than average percentage runoff expected from these surfaces. Unfortunately, in some PMF calculations, the rural percentage runoff already exceeds 70%. Thus the effect of applying the urban adjustment would be to *reduce* PR values. This is clearly anomalous. In such cases it is preferable to omit the adjustment for urbanisation from the PR calculation.

Anomaly 2: "First it passes, now it fails"
A second anomaly concerns PMF calculations for reservoirs 'in cascade'. The existing spillway arrangements at the upper reservoir are checked in the normal way. Let us suppose that the dam can just safely pass the PMF. When we consider flood calculations for the lower reservoir — using the procedure given in FSSR 10 — we find that the reservoir is sensitive to a longer duration storm (than the upper reservoir). This is partly because of the longer distances that runoff has to travel and partly because of the additional attenuation and delaying effects imposed by the second reservoir.

The PMF procedure provides that the maximum rainfall of duration D (the design storm duration) should contain within it the maximum rainfall of all shorter durations. Thus the design storm used in calculations for the lower reservoir is every bit as intense as that used in calculations for the upper. The one difference is that the storm is longer and therefore comprises a greater storm depth. The greater storm depth in turn leads to a slightly higher percentage runoff.

In order to allow for the routing effect of the upper reservoir it is of course necessary to apply the design storm first to the catchment to the upper reservoir alone (routing it through the upper reservoir), then to the intervening catchment (i.e. the catchment local to the lower reservoir). Subsequently, the routed outflow from the upper reservoir is combined with the synthesised runoff from the intervening catchment before finally routing this through the lower reservoir. This provides the required check of the spillway facilities at the lower reservoir.

The anomaly is that, during the process, the flood passing through the upper reservoir will be greater than the PMF previously estimated; perhaps the spillway facilities will now fail. When first encountered, the phenomenon of "first it passes, now it fails" is both surprising and unsettling. However, when it is recognised for what it is — a spurious effect that arises from the pessimistic assumption to 'nest' the worst maximum rainfall of each duration within a single design storm — it can be ignored. This anomaly does not arise in T-year flood calculations.

Further practical matters concerning reservoir flood calculations are discussed in Section 5.3.

2.4 Use of local data

There is now fairly wide recognition that the types of hydrological models presently available for general use provide only coarse estimates of everyday flood events. That they should provide adequate estimates of extreme events is, more often than not, an act of faith. Boorman *et al.* (1990) found that the FSR rainfall-runoff model was no exception to this rule. The model provides relatively poor estimates of flood frequency on many catchments, unless recourse is made to locally observed flood data. Thus the recommendations to consider local data, made in the FSR and strengthened in FSSR 13, must remain.

The application of this recommendation to reservoir flood safety assessment causes some difficulty. It is easy for the researcher to recommend special monitoring of reservoired catchments. But there are many catchments to monitor and the installation and operation of such equipment may be expensive. Should such monitoring have a higher priority than monitoring piezometric levels within an embankment?

A particular problem is that, if local flood data indicate a lower than 'normal' response, the engineer may be reluctant to extrapolate the observation to reduce estimates of the PMF. Yet if the local data — no matter how sparse — indicate a higher than 'normal' response, how can the engineer possibly neglect to act on it? Thus the use of local data can be one-sided in reservoir applications.

If the hydrologist overemphasises the weakness of flood estimation procedures based on a rainfall-runoff model generalised in terms of half a dozen catchment characteristics, this might encourage the engineer to move back to simpler methods which rely more on judgement than on models, rather than to move forward through local instrumentation.

However, if the Panel Engineer states that a particular reservoired catchment should be instrumented, the reservoir owner will have little option but to comply with the request. It should be recognised that there have been significant advances in data logging systems in recent years, both in terms of improved reliability and automated data processing. A coordinated initiative would, in time, yield extensive flood data for typical upland reservoired catchments, and there might then be scope for the research hydrologist to provide an improved generalisation of flood runoff on such catchments. It is conceivable that the flood hydrology of such catchments (of a generally impervious and undeveloped nature) may be strikingly less variable than that of the largely lowland catchments on which the procedures are presently based.

Regional variations
Some important regional variations in estimation procedures are summarised in Appendix B3.

2.5 Why the statistical method is not recommended

Documents A and B imply that a direct statistical method of flood estimation should not be used for reservoir safety application, but the reasons are not stated prominently.

It is convenient to argue that a design hydrograph is always required and that this calls for the use of a rainfall-runoff method. However, there are several ways in which flood estimates from different sources can be reconciled (see FSSR 13; Reed, 1987). For example, a rainfall-runoff model parameter such as standard percentage runoff (SPR) might be adjusted so that the flood frequency relationship tallied with a statistical analysis of peak flows. Alternatively it is possible to exploit FSSR 9 to 'flesh out' a peak flow estimate to provide a design hydrograph (Reed, 1987).

However, there is a second reason why present guidance discourages the use of the statistical method. If estimates are tailored closely to local data, there is the danger that extrapolation to the high return periods relevant to reservoir flood design may lead to gross under- or over-design. This fear also attends the use of local data in the rainfall-runoff method (see Section 2.4), but because of the greater regional homogeneity in extreme rainfall and the longer record lengths available for analysis, use of the rainfall-runoff method is preferred for reservoir safety applications. A further reason for preferring a rainfall-runoff method may be that using a rainfall-runoff method is in some sense more 'supportable', since it is based on a conceptual model of flood formation rather than on statistics alone.

For completeness, Appendix B refers to both the rainfall-runoff and the statistical methods of flood estimation.

3 Catchments

3.1 Gauged catchments on which the current procedure is based

Very few of the gauged catchments analysed in the FSR are reservoired catchments or similar to reservoired catchments. Boorman (1985) considers a more extensive data set when deriving revised estimation equations for the rainfall-runoff method. The range of catchment types is summarised in Table 3.1 by reference to values of seven characteristics for the 209 catchments analysed by Boorman.

3.2 Typical reservoired catchments

In order to examine the extent to which typical reservoired catchments differ intrinsically from typical gauged catchments, a selective survey of reservoir flood applications was carried out.

Several water authorities and consulting engineers were asked to provide information relating to actual reservoir applications of the FSR flood estimation procedures. The survey focused on reservoirs where the Probable Maximum Flood (PMF) inflow had been evaluated. Reservoirs were excluded from the survey if they were affected by other reservoirs sited upstream.

The users supplied standard FSR characteristics for the survey catchments: AREA, SAAR, RSMD, S1085, MSL, URBAN, and SOIL. They also supplied the peak PMF inflow and details of the method used in its derivation. The summary information given for reservoired catchments in Table 3.1 is based on a sample size of 187, of a similar order of magnitude to the sample of 209 gauged catchments.

3.3 Comparison of reservoired and gauged catchments

Comparison of the median catchment characteristics given in Table 3.1 indicates that SAAR, RSMD and SOIL values for gauged and reservoired catchments are numerically similar. These characteristics are typically a little higher for the reservoired catchments, reflecting their predominant location in upland, impermeable areas. Nevertheless, there is considerable commonality in these characteristics between the gauged and reservoired catchments.

In contrast, values of AREA, S1085 and MSL show marked differences. The median value of AREA for the reservoired catchments is almost 30 times smaller than for the gauged catchments. A significant difference is also evident in mainstream lengths and slopes: the reservoired catchments are typically more than five times steeper than the gauged catchments.

Table 3.1 *Comparison of characteristics: g = gauged catchments (sample size 209), r = reservoired catchments (sample size 187)*

Name	Unit		Mean	Median	Minimum	Maximum	Meaning
AREA	km^2	g:	142.7	98.8	0.04	616.4	catchment area
		r:	11.9	3.48	0.04	256.0	
MSL	km	g:	22.4	19.6	0.16	84.6	mainstream length from 1:25000
		r:	3.38	2.00	0.09	35.4	OS map
S1085	$m\ km^{-1}$	g:	12.3	6.39	0.92	179.7	mainstream slope between sections
		r:	52.4	36.0	2.01	252.1	10 and 85% from catchment outlet
SAAR	mm	g:	1192	1024	559	3596	standard period average annual
		r:	1244	1137	600	3050	rainfall (1941-1970)
RSMD	mm	g:	43.2	39.4	17.3	107.5	effective 1-day maximum rainfall of
		r:	45.0	43.6	22.1	94.0	5-year return period
SOIL	-	g:	0.403	0.426	0.15	0.50	index of winter rainfall acceptance
		r:	0.450	0.480	0.15	0.50	potential
URBAN	-	g:	0.056	0.00	0.00	0.81	urban fraction of catchment from
		r:	0.014	0.00	0.00	0.75	1:50000 OS map

While these differences are not too surprising, they underline the act of faith that is necessary in applying a rainfall-runoff method calibrated on one data set to the typical reservoir applications highlighted by the other one. Because of the topographic differences, there is particular concern about the estimation of catchment response times. The revised method of synthesising Tp(0) rather than Tp (Boorman, 1985) gives hope for improved extrapolation of the estimation equation to very quickly responding catchments (see also Reed, 1985), but concern remains that response times may not be well estimated on the small, steep catchments with shallow soils which are typical of reservoir applications. Another form of comparison is provided in Table 3.2 which summarises the extent to which physical and climatic characteristics are cross-correlated within each data set.

The cross-correlations for the reservoired data set are generally somewhat weaker than for the gauged catchments. This might be taken to imply a slightly richer diversity within the reservoired catchment data set (contradicting the views expressed in Section 2.4). However, the weaker correlations may simply reflect the greater difficulty in deriving catchment characteristics consistently on smaller catchments from the available maps.

3.4 Selection of case study reservoired catchments

During the study a contract revision was agreed to include a comparison of T-year flood and PMF estimates on reservoired catchments (see paragraph 1a in Appendix A). In order to assess implications more fully, it is necessary to compare them in terms of reservoir peak outflows. Rather than carrying out a further extensive survey of reservoir applications, it was decided to concentrate on comparing only a subset of reservoirs. This was selected carefully so as to retain a representative sample of reservoired catchments.

The selection was carried out by categorising the 187 reservoired catchments according to values of AREA, S1085 and SAAR. The 187 values for AREA, S1085 and SAAR were ranked and the values divided into three bands of equal size: those with small, medium and large values of the characteristic. The three-way classification of each of three characteristics results in an overall division into 27 categories, as defined in Table 3.3 and illustrated in Figure 3.1.

The numbers shown in Figure 3.1 are counts of the number of reservoirs which fall into each category. Some categories have only one

Table 3.2 *Cross-correlation matrices*

gauged catchments (sample size 209)

	ln1+URBAN	lnSOIL	lnRSMD	lnSAAR	lnS1085	lnMSL
lnAREA	-0.11	-0.20	-0.21	-0.20	-0.66	0.95
lnMSL	-0.14	-0.11	-0.16	-0.15	-0.61	
lnS1085	-0.15	0.34	0.72	0.70		
lnSAAR	-0.31	0.39	0.97			
lnRSMD	-0.30	0.39				
lnSOIL	-0.13					

reservoired catchments (sample size 187)

	ln1+URBAN	lnSOIL	lnRSMD	lnSAAR	lnS1085	lnMSL
lnAREA	0.06	-0.04	-0.09	-0.13	-0.53	0.90
lnMSL	0.07	-0.09	-0.14	-0.18	-0.50	
lnS1085	-0.17	0.16	0.52	0.50		
lnSAAR	-0.24	0.41	0.92			
lnRSMD	-0.26	0.32				
lnSOIL	-0.07					

Table 3.3 *Classification scheme for reservoired catchments*

AREA	km^2	"small"	<	1.7	<	"middling"	<	6.8	<	"large"
SAAR	mm	"dry"	<	995	<	"moderate"	<	1422	<	"wet"
S1085	m km^{-1}	"flat"	<	23	<	"medium"	<	59	<	"steep"

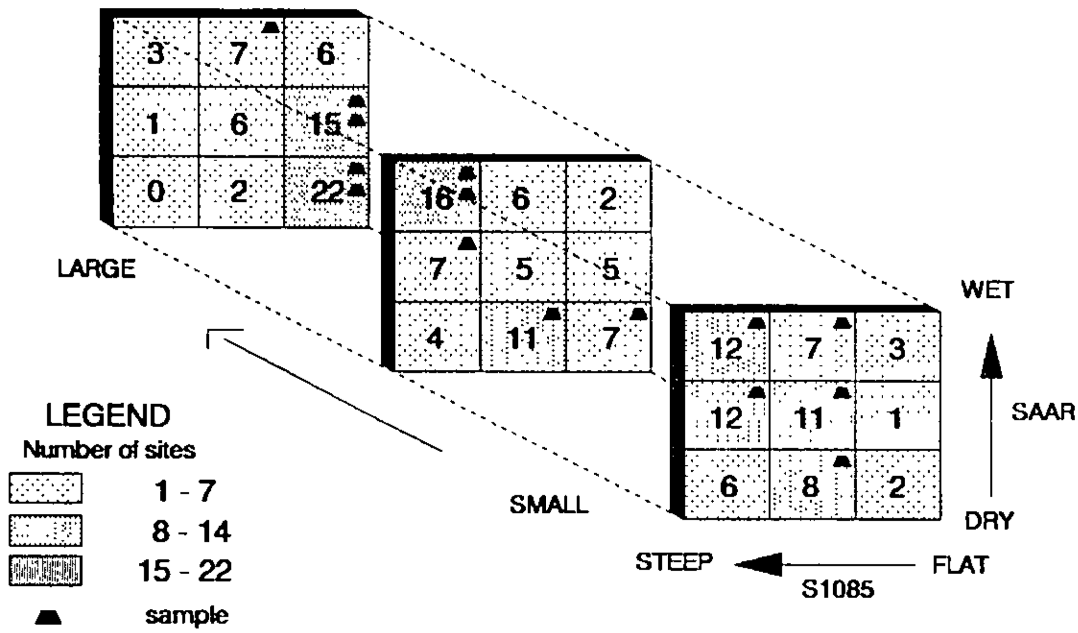


Figure 3.1 *Selection of representative reservoired catchments for case study*

member, e.g. the one 'small flat catchment in a moderate rainfall area' amongst the 187 reservoired catchments surveyed is Brother Loch, near Glasgow.

Fifteen case study catchments were selected by choosing one catchment from any category having seven to 14 members, and two catchments from any category having 15 or more members. Within this framework, catchments were selected at random. The resultant set of case study reservoired catchments is detailed in Table 3.4. Their locations relative to hydrometric areas are shown in Figure 3.2. It should be noted that all of the catchments have an urban fraction (URBAN) of zero. The majority of major impounding reservoirs in the UK drain largely or entirely rural catchments. (Particular care should be exercised when estimating floods in exceptional cases where the catchment is partially or heavily urbanised; such cases might include former supply reservoirs that have been swallowed up by urban development, or major flood-balancing ponds.)

It transpired that this careful selection procedure did not yield an entirely typical set of example catchments. Kirbister is a raised loch in the Orkney Islands; Loch Craisg and Parkhill House are also considered unusual (personal communication from McKenna, see acknowledgements, p. 58), but it was decided not to make any late changes to the set of catchments analysed.

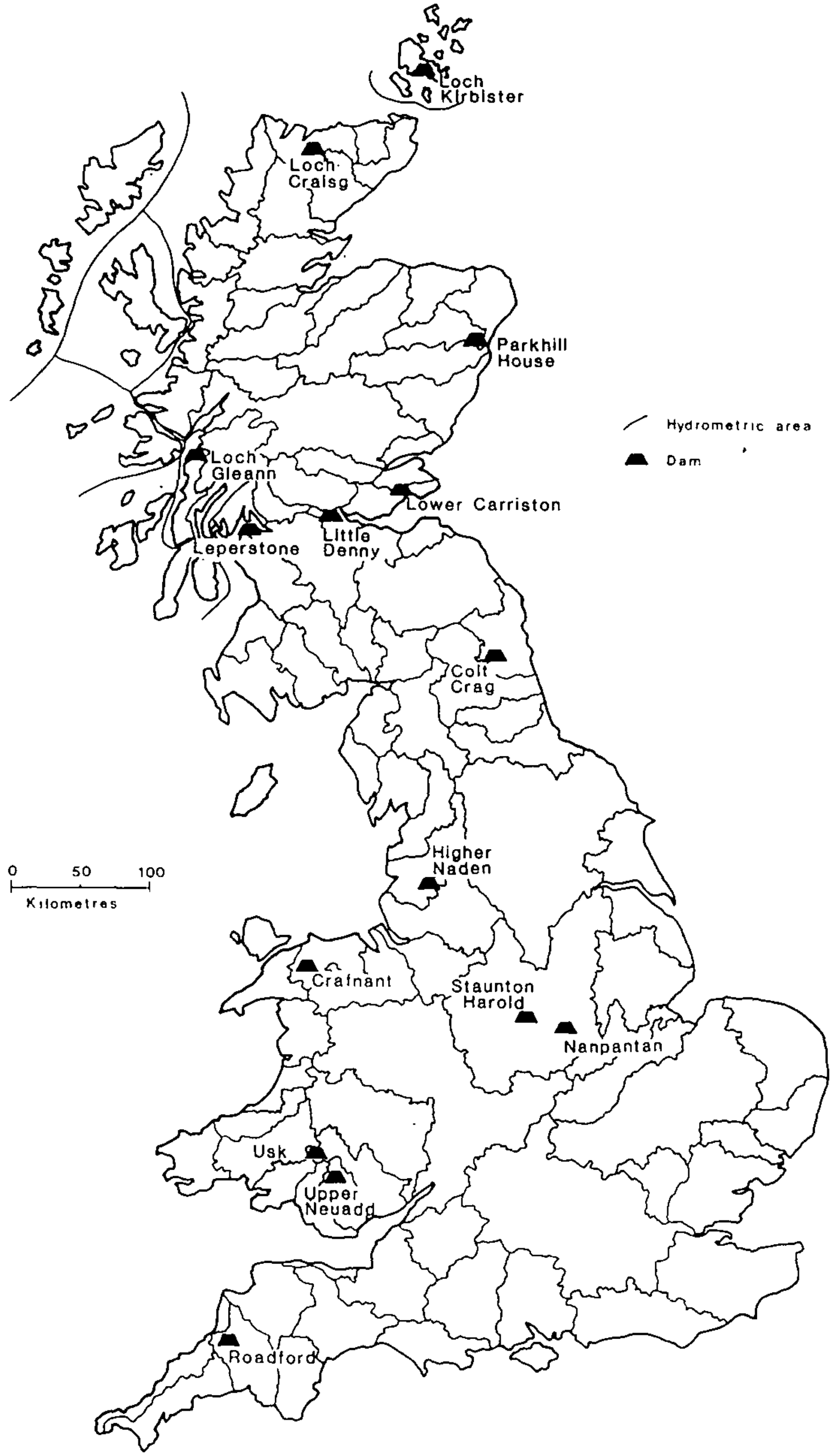


Figure 3.2 *Location of case study reservoired catchments listed in Table 3.4, showing hydrometric areas*

Table 3.4 Characteristics of case study reservoired catchments

Name		Grid ref.	AREA	MSL	S1085	SOIL	SAAR	M5-2D	r	Em-2h	EM-24h
			km^2	km	$m\ km^{-1}$	-	mm	mm	-	mm	mm
Loch Craisg	NC	599579	0.74	0.80	95.0	0.50	1090	63	0.20	115	250
Little Denny	NS	793810	0.98	0.75	30.2	0.45	1200	61	0.27	137	250
Loch Gleann	NM	850273	1.21	1.10	69.09	0.40	1620	78	0.17	121	300
Parkhill House	NJ	894141	1.21	1.40	31.43	0.30	825	62	0.25	126	245
Leperstone	NS	354720	1.22	0.60	40.0	0.45	1540	68	0.25	131	275
Higher Naden	SD	852165	3.90	2.00	96.0	0.50	1400	74	0.26	165	290
Lower Carriston	NO	328036	3.94	3.30	34.30	0.45	820	57	0.27	135	247
Nanpantan	SK	508172	4.28	2.50	19.5	0.45	725	53	0.39	180	270
Upper Neuadd	SO	030188	5.74	2.54	106.0	0.50	2300	125	0.16	160	400
Crafnant	SH	745608	6.20	3.30	101.0	0.40[1]	1980	108	0.20	138	320
Usk	SN	806280	13.50	4.80	40.0	0.50	1700	94	0.23	121	300
Colt Crag	NY	946777	18.05	7.45	10.4	0.50	800	50	0.32	150	290
Loch Kirbister	HY	369073	20.73	4.30	17.98	0.40[1]	1100	53	0.22	115	240
Staunton Harold	SK	380242	26.30	4.25	9.40	0.41[2]	700	49	0.40	177	270
Roadford	SX	421900	34.69	5.16	18.6	0.44[3]	1245	70	0.29	167	326

Note: All catchments have an urban fraction of zero (URBAN=0)

Footnote on mixed soil types (i.e. WRAP classes):
[1] 50% type 2, 50% type 5
[2] 27% type 2, 73% type 4
[3] 7% type 2, 93% type 4

3.5 Selection of case study gauged catchments

Standard analyses for the 209 gauged catchments in the Institute of Hydrology's flood event archive have already been carried out (Boorman, 1985) and they are not repeated here.

However, reference is made in Chapters 7 and 8 to specific gauged catchments which relate to particular objectives of the project: their leading characteristics are summarised in Table 3.5.

In comparison to typical UK gauged catchments, all but the Alston catchment are relatively small upland catchments and might initially be thought to be broadly typical of reservoired catchments in the Pennine region, where many of the UK's oldest dams are sited. However, reference to Table 3.3 shows that — in comparison to the 187 reservoired catchments surveyed — the catchments are all 'large, wet and of medium slope'. Thus they are only typical of a subset of UK reservoired catchments. It was not practical to investigate any further catchments within the project.

Table 3.5 Case study gauged catchments

Name	AREA	S1085	SAAR
	km^2	$m\ km^{-1}$	mm
Bottoms Beck at Bottoms	10.6	30.8	1461
Croasdale Beck at Croasdale	10.4	37.8	1839
Harwood Beck at Harwood	25.1	29.9	1669
Langdon Beck at Langdon	13.0	26.6	1621
South Tyne at Alston	118.5	23.9	1438
Troutbeck at Moorhouse	11.4	35.8	2182

4 Comparisons of flood estimates: excluding reservoir effects

4.1 PMF_{rapid} v PMF_{full}

Background
The full method of estimating the Probable Maximum Flood (PMF) requires convolution of the Probable Maximum Precipitation (PMP) — after subtraction of losses — with a unit hydrograph, and addition of a baseflow allowance. Options exist for seasonal estimates of PMP, including allowances for snowmelt and for increased runoff from frozen ground in the winter.

Document D (see Section 2.1) provides a short-cut which approximates to the PMF inflow peak. This was based on calculations of PMF by the full method, combining the 'all-year' PMP with the snowmelt allowance but omitting any allowance for frozen ground. PMF values were calculated for an 80-catchment subset of those used in the development of the FSR rainfall-runoff model. The values were used to derive the regression equation:

$$PMF = 0.835\ AREA^{0.878}\ RSMD^{0.724}\ SOIL^{0.533}\ (1+URBAN)^{1.308}\ S1085^{0.162} \qquad 4.1$$

Analysis
The faithfulness of the method was checked by reference to the full sample of 187 reservoired catchments gathered in the PMF survey. These are grouped for convenience into national plots. From Figure 4.1 it can be seen that the rapid method generally provides an excellent approximation, amply justifying the claim that the method is adequate for preliminary screening purposes.

The opportunity was taken in the present study to derive revised regression equations based on the sample of 187 reservoired catchments. Step-wise regression of PMF on the independent variables AREA, RSMD, SAAR, SOIL, URBAN and S1085 yielded the equation:

$$PMF = 0.881\ AREA^{0.925}\ S1085^{0.291}\ RSMD^{0.547}\ (1+URBAN)^{2.16} SOIL^{0.472} \qquad 4.2$$

This explains 98.0% of the variation in lnPMF, with a factorial standard error of 1.21. When RSMD was eliminated from the analysis, the alternative equation:

$$PMF = 0.629\ AREA^{0.937}\ S1085^{0.328}\ SOIL^{0.471}\ (1+URBAN)^{2.04} SAAR^{0.319} \qquad 4.3$$

explained 97.6% of the variation in lnPMF, with a factorial standard error of 1.23. Use of Equation 4.3 avoids the fairly lengthy calculation of the RSMD variable.

Limitations
Although the rapid method provides a good initial estimate of the PMF peak inflow, the full method needs to be used to obtain the complete inflow hydrograph for subsequent routing through the reservoir. There are also circumstances under which this particular variant (all-year PMP plus snowmelt but excluding frozen ground) may be inappropriate. Since the publication of Document A, it has become usual to distinguish between summer and winter estimates of PMF (see Section 4.3).

4.2 PMF v NMF

The Normal Maximum Flood (NMF) formed the linchpin of reservoir flood estimation in the UK prior to the publication of the Flood Studies Report. The NMF is defined in terms of catchment area and is conveniently summarised by a plot of ln(NMF/AREA) against lnAREA. (See solid lines in Figure 4.2). For structures requiring a high degree of safety, a design flood — usually referred to as the 'catastrophic flood' — was taken as a multiple of the NMF, a factor of two being typical.

Reservoir flood standards and estimation methods have moved on, but it is instructive to compare past and present methods for the set of 187 reservoired catchments. Figure 4.2 confirms that the PMF inflow to a reservoir is far from being a simple function of catchment area. Moreover it shows clearly that the new standard does not lead to larger design floods in every case. While there are some instances where PMF/NMF exceeds 3.0, there are others where it falls short of 1.0. Thus the analysis does not support the assertion that the FSR procedure has unilaterally increased the magnitude of design floods. Rather it has *changed* the magnitude of design floods.

It is suggested in passing that the increased expenditure on spillway improvements since the publication of Document A reflects two principal factors. Firstly, many small reservoirs did not meet pre-existing standards. Secondly, the PMF procedure is a radically different type

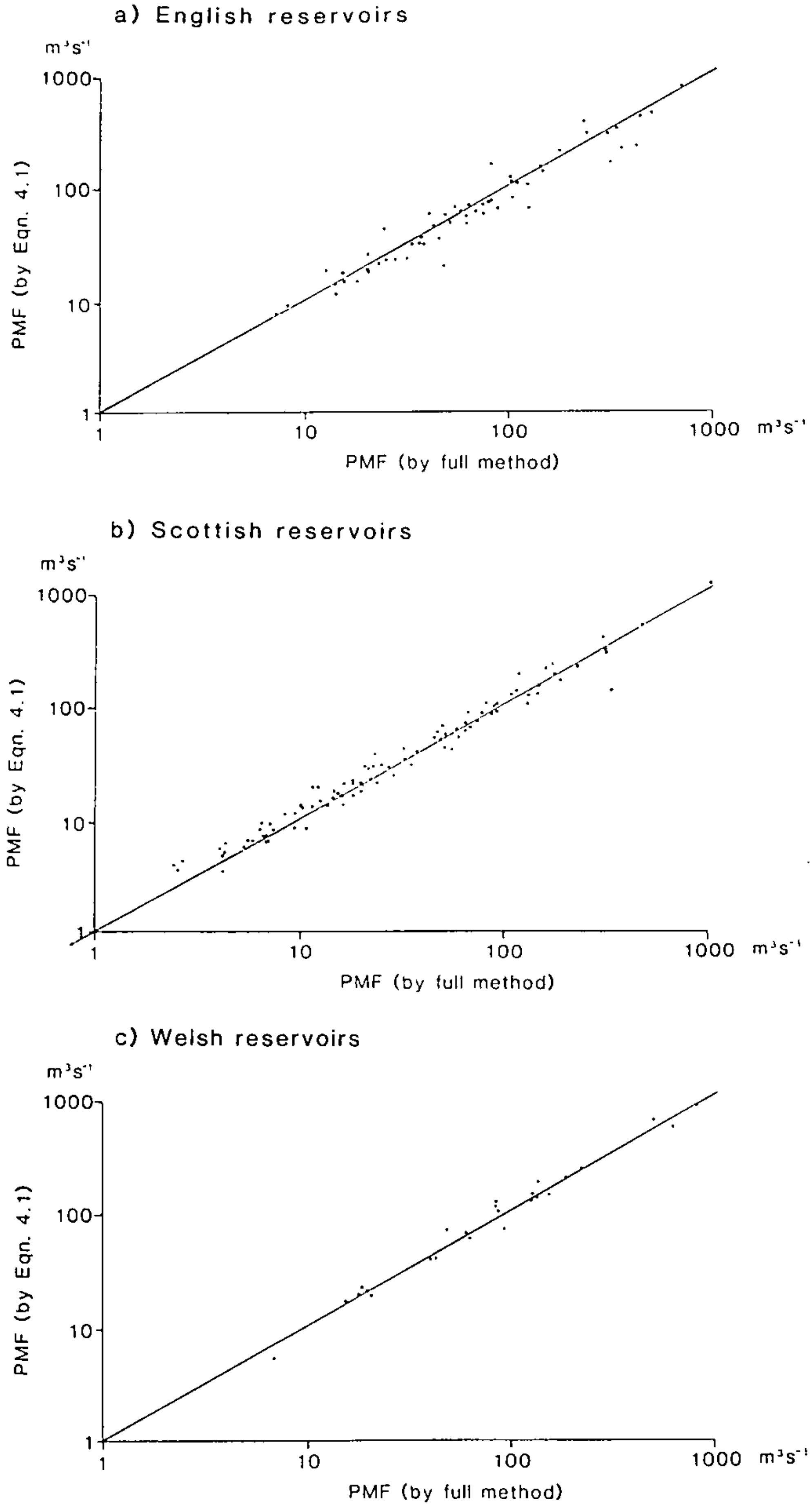


Figure 4.1 *Comparison of estimates of PMF by Equation 4.1 with estimates by full method*

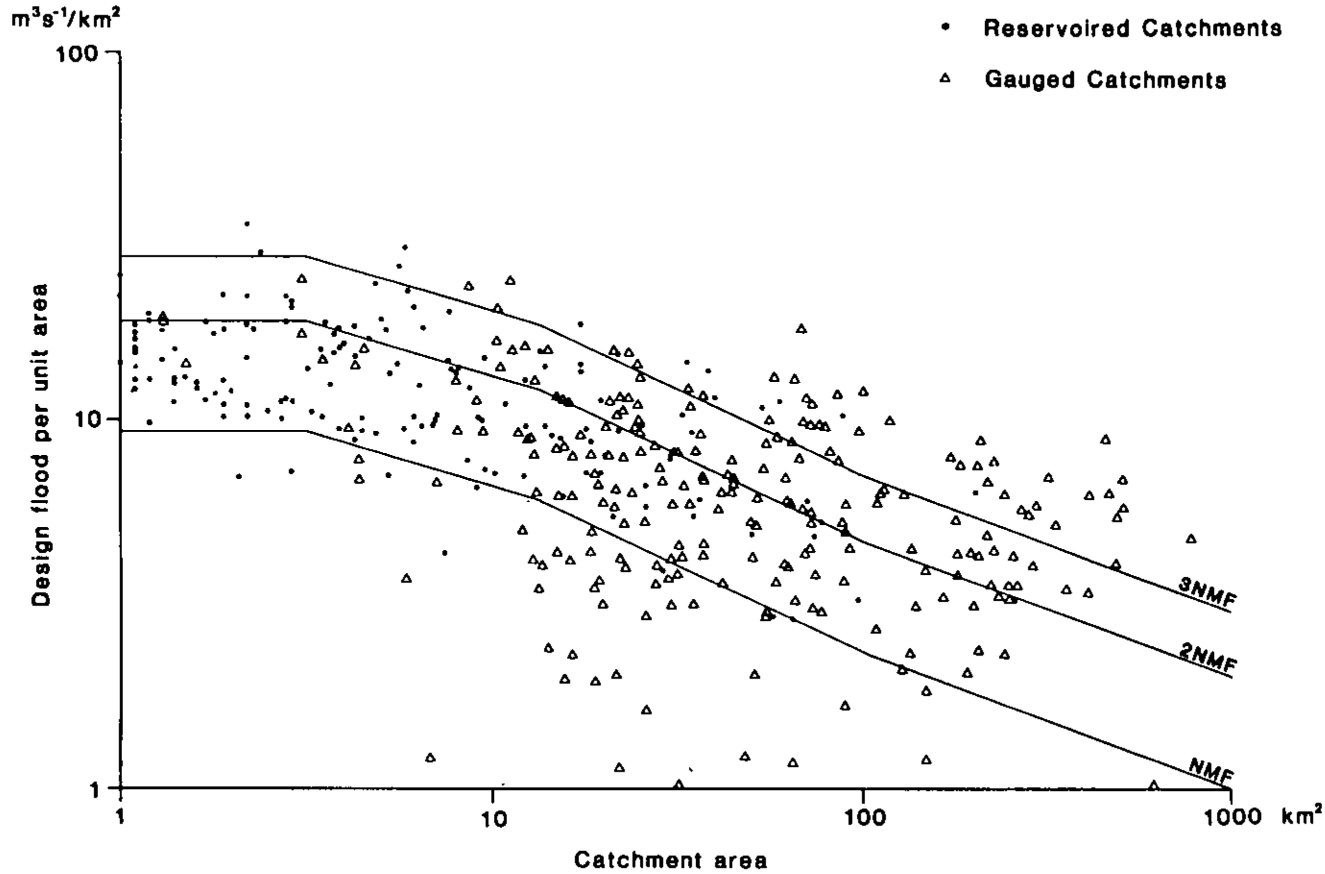


Figure 4.2 *Design floods per unit area — symbols denote calculated PMFs*

of flood estimation method from the 'envelope' method that it largely replaced. Appreciable capital expenditure has been incurred where the new procedure has indicated a higher spillway capacity, but there has been no financial rebate where the new procedure has indicated a smaller required spillway capacity.

4.3 PMF_{summer} v PMF_{winter}

In accordance with recommendations in Document A, summer and winter estimates of PMF are generally considered separately, in order to ascertain which season produces the higher design requirement.

Table 4.1 indicates the magnitude of the PMF, the season supplying it, and the relative magnitude of the 'low-season' PMF. Computation of the winter value included the 'usual' allowances for snowmelt (1.75 mm h^{-1} for part or all of the design event) and frozen ground (soil type set to WRAP class 5). It is seen that, with one exception, the summer season provides the PMF. Loch Gleann is a moderately permeable catchment for which the frozen ground assumption makes the winter value surpass the summer value. By cross-reference to Table 3.4 it can be confirmed that the summer dominance is less pronounced in areas of high average annual rainfall, reflecting the seasonal variation in PMP (see also Section 6.1).

It will be shown in Section 5.2, for the 15 example catchments, that the apparent dominance of the summer case is much reduced when allowance is made for reservoir effects.

4.4 PMF v T-year flood

The reservoir flood safety standards recommended in Document A stipulate that earthen embankment dams sited above a community should be designed to pass the Probable Maximum Flood. However, Table 1 of Document A indicates that design floods less than the PMF can be used for dams where a breach would pose less of a threat to life or where the structure is designed to withstand some degree of overtopping without breaching.

Table 4.1 Comparison of summer and winter values of PMF, excluding reservoir effects

Name	Probable Maximum Flood $m^3 s^{-1}$	Season of PMF S: May - Oct W: Nov - Apr	Summer value as % of PMF	Winter value as % of PMF
Loch Craisg	10.4	S	100	76
Little Denny	12.5	S	100	77
Loch Gleann	16.5	W	94	100
Parkhill House	8.4	S	100	80
Leperstone	17.8	S	100	93
Higher Naden	75.6	S	100	68
Lower Carriston	33.9	S	100	64
Nanpantan	40.9	S	100	53
Upper Neuadd	133.3	S	100	90
Crafnant	95.1	S	100	99
Usk	217.4	S	100	92
Colt Crag	127.2	S	100	56
Loch Kirbister	133.2	S	100	95
Staunton Harold	166.4	S	100	64
Roadford	377.8	S	100	85

Table 4.2 Ratio of T-year flood to PMF, excluding reservoir effects

Name	Q_{150} : PMF	Q_{1000} : PMF	Q_{10000} : PMF
Loch Craisg	0.19	0.29	0.50
Little Denny	0.19	0.28	0.47
Loch Gleann	0.16	0.24	0.42
Parkhill House	0.17	0.26	0.45
Leperstone	0.21	0.30	0.51
Higher Naden	0.20	0.30	0.54
Lower Carriston	0.18	0.26	0.44
Nanpantan	0.16	0.25	0.45
Upper Neuadd	0.23	0.34	0.59
Crafnant	0.26	0.39	0.70
Usk	0.22	0.32	0.56
Colt Crag	0.17	0.25	0.44
Loch Kirbister	0.17	0.25	0.43
Staunton Harold	0.15	0.22	0.39
Roadford	0.18	0.26	0.46
geometric mean:	0.19	0.28	0.48
nominal ratio:	0.20	0.30	0.50

In these cases Document A provides two sets of design standards. One set is phrased as factors of the PMF (namely 0.5PMF, 0.3PMF and 0.2PMF), while the other is set in terms of flood return period (namely 10,000-year flood, 1000-year flood and 150-year flood). While Table 1 of Document A indicates that the higher value should be used, it is evident (Clarke & Phillips, 1984) that many Panel Engineers have a strong personal preference. Each set has its adherents and it seems that the choice of one or the other is partly a matter of philosophy and partly a matter of convenience. Some comparisons are therefore of interest and are given in Table 4.2.

On that evidence it would appear that the two sets of standards laid down in Document A were well chosen to be broadly consistent. Taking the Q_{10000} : PMF comparison as an example, those having the lowest ratio would appear to be permeable or low-SAAR catchments, while those having the highest ratio are high-SAAR catchments in Wales.

Because reservoir routing effects are excluded from the comparison, Table 4.2 does not tell the whole story. Corresponding results which do include reservoir routing effects are given in Table 5.8.

4.5 Sensitivity to design storm duration

The choice of design storm duration, D, within the FSR rainfall-runoff method is made by reference to:

$$D = (1 + SAAR/1000)\ Tp \qquad 4.4$$

where SAAR is standard average annual rainfall (mm) and Tp is the unit hydrograph time-to-peak. Both D and Tp are expressed in hours.

Table 4.3 *Comparison of D_{crit} and D_{FSR}, excluding reservoir effects (all durations in hours)*

Name	D_{FSR}	Return period (years) 10 D_{crit}	100 D_{crit}	1000 D_{crit}	10000 D_{crit}
Loch Craisg	3.25	3.25 / 4.25	3.75 / 4.25	3.75	3.25 / 3.75
Little Denny	4.25	3.75 / 4.75	4.25 / 4.75	4.25	3.75
Loch Gleann	3.75	3.75 / 4.25	4.25 / 4.75	4.25	3.75
Parkhill House	5.5	7.5 / 9.5	6.5 / 7.5	5.5 / 6.5	5.5
Leperstone	3.75	3.25 / 3.75	3.25 / 3.75	3.25	3.25
Higher Naden	3.75	4.25	3.75	3.25 / 3.75	2.75
Lower Carriston	6.5	8.5	7.5	6.5	6.5
Nanpantan	6.5	7.5	6.5	5.5 / 6.5	5.5
Upper Neuadd	4.25	4.25	3.75	3.25	2.75
Crafnant	4.25	4.25	3.75	3.25	3.25
Usk	6.25	5.75 / 6.25	5.25	4.75	4.25
Colt Crag	11.	13.	11.	11.	9.
Loch Kirbister	7.5	9.5	9.5	8.5	8.5
Staunton Harold	11.	11.	11.	9.	9.
Roadford	8.5	8.5	7.5	7.5	6.5
geometric mean:	5.55	6.12	5.72	5.21	4.76
as % of D_{FSR} value:	100	110	103	94	86

Table 4.4 *$Q_T(D_{FSR})$ as percentage of $Q_T(D_{cnt})$, excluding reservoir effects*

Name	Return period, T years			
	10	100	1000	10000
Loch Craisg	100	99	100	100
Little Denny	100	100	100	100
Loch Gleann	100	99	100	100
Parkhill House	98	99	100	100
Leperstone	100	100	100	99
Higher Naden	100	100	100	99
Lower Carriston	99	100	100	100
Nanpantan	99	100	100	100
Upper Neuadd	100	100	99	98
Crafnant	100	100	99	98
Usk	100	99	98	97
Colt Crag	99	100	100	100
Loch Kirbister	99	99	99	100
Staunton Harold	100	100	100	99
Roadford	100	100	99	98

The design storm duration is invariant with return period in the unreservoired case. This standard storm duration, denoted here by D_{FSR}, was compared with other trial values of D to establish the extent to which Equation 4.4 captures the storm duration yielding the highest flood peak. In what follows, D_{cnt} denotes the storm duration yielding the highest peak flow.

The results are summarised in Table 4.3. The FSR method insists that the design storm should be chosen to be an odd multiple of the modelling interval, which should itself be chosen to be a convenient interval approximately equal to Tp/5. The modelling intervals used in the fifteen case studies can be inferred from the Table: eight of 0.25 h, five of 0.5 h and two of 1 h.

All the results presented in Chapters 4 and 5 have been derived using Version 2 of the Micro-FSR computer package. Where Table 4.3 gives two values of D_{cnt}, this is because the output format did not permit precise establishment of the duration yielding the highest outflow peak: the true value of D_{cnt} will be within the range indicated. The first point to note is that none of the D_{cnt} values differs greatly from the standard D_{FSR} value. In relative terms the greatest departures are for Upper Neuadd and Usk at the 10,000-year return period. However, the resultant flood peaks are scarcely affected by the difference between D_{FSR} and D_{cnt}, as confirmed by Table 4.4.

It is instructive to note from the summary information at the foot of Table 4.3 that D_{cnt} decreases systematically with increasing return period. The effect is thought to stem from rainfall frequency behaviour: the distribution of maximum D-hour rainfall depths is increasingly skewed at shorter durations, so that shorter durations tend to have steeper rainfall growth curves (e.g. Dales & Reed, 1989).

The value of D_{FSR} given by Equation 4.4 is generally within the range of the D_{cnt} values. Typically, D_{FSR} and D_{cnt} correspond at about the 200-year design flood event.

5 Comparisons of flood estimates: including reservoir effects

5.1 Sensitivity to design storm duration

In reservoired applications, the design storm duration is extended by adding the reservoir response time, RLAG, to the catchment response time, Tp. Thus:

$$D = (1 + SAAR/1000)\ (Tp + RLAG) \qquad 5.1$$

where RLAG is defined as the time lapse (in hours) between peak inflow to the reservoir and peak outflow from it. FSSR 10 sets out a method for calculating flood estimates for reservoirs in cascade, which extends the definition of this equation further.

Concern has been expressed that Equation 5.1 may sometimes fail to capture the storm duration to which the catchment-reservoir system is typically most sensitive. Parallel advice for the design of flood balancing ponds (Hall & Hockin, 1980; Hall *et al.*, in preparation) is to consider a range of storm durations and, implicitly, to adopt the one which yields the highest peak water level.

This appears to be a legitimate concern in those complex mixed urban-rural cases where it is unclear whether the 'slow' rural response or the 'fast' urban response dominates. The locations of the urbanisation and the balancing pond within the overall catchment to the critical site (which the reservoir is intended to protect) may be particularly important. By their nature, balancing ponds are intended to hold back and attenuate floods rather more specifically than impounding reservoirs do. Heavily throttled outlet devices are common, so it is to be expected that the design of balancing ponds will be rather more sensitive to design storm duration. Only the design of impounding reservoirs is considered in this report.

In the reservoired case, the presence of RLAG in Equation 5.1 means that the design storm duration is not known *a priori*. Three or four iterations usually suffice to determine D_{FSR}. This notation signifies that the standard FSR approach has been used to determine design storm duration, but in the reservoired case this is by Equation 5.1 not 4.4. Because of reservoir storage effects, D_{FSR} is no longer invariant with return period and comparisons are rather more complicated than for the unreservoired case.

The label 'OG' in Table 5.1 *et seq.* indicates a reservoir for which the spillweir discharge-head relationship used is of the conventional 'ogee' type:

$$q = bc\ (h-d)^{1.5} \qquad 5.2$$

using the notation defined in Chapter 9 (which considers the theory of reservoir routing). Results for other reservoirs may be influenced by particular characteristics of the assumed discharge-head relationship and should therefore be interpreted with caution. The relevant characteristics of the 15 case study reservoirs are given in Appendix D.

For brevity, results are presented only for 100-year and 10,000-year designs (Tables 5.1 and 5.2 respectively). It can be seen from Table 5.1 that D_{cnt} is sometimes much longer than D_{FSR} and that this can have a marked effect on the resultant flood peak, as for the 100-year flood at Loch Gleann.

Conversely, in Table 5.2 it is seen that D_{cnt} is sometimes rather shorter than D_{FSR}, as for the 10,000-year flood at Upper Neuadd. However, in most cases, the flood peak resulting from the standard FSR procedure is within a few per cent of that derived using the 'worst case' approach.

5.2 Sensitivity to reservoir characteristics

The process of 'routing' a flood through a reservoir is considered in detail in Chapter 9. The effect of a reservoir is to delay and attenuate (i.e. reduce the amplitude of) the flood hydrograph from the catchment. It is generally advisable for the routing procedure to make *explicit* allowance for rain falling directly onto the reservoir: all results presented in this chapter are based on this approach.

The reservoir lag time, RLAG, is primarily determined by the storage-discharge characteristics of the reservoir. Flood magnitude also has some influence: the lag is generally rather less for rarer events (see Table 5.3). Parkhill House is something of an exception because the discharge-head relationship shows extreme throttling of the rarer events.

Table 5.1 *Comparison of D_{FSR} and D_{crit} for 100-year flood, including reservoir effects*

Name		D_{FSR} *h*	D_{crit} *h*	$Q_{100}(D_{FSR})$ as % of $Q_{100}(D_{crit})$
Loch Craisg	OG	6.25	9.75 / 10.25	96
Little Denny	OG	10.25	14.75	99
Loch Gleann	OG	12.75	c 35.5	89
Parkhill House		10.5	c 19.5	98
Leperstone	OG	9.25	12.75	99
Higher Naden		5.25	4.25	99
Lower Carriston	OG	8.5	9.5 / 10.5	100
Nanpantan	OG	8.5	7.5	100
Upper Neuadd	OG	7.75	7.25	100
Crafnant		9.75	13.25	98
Usk		5.75	18.25	100
Colt Crag	OG	17.	17.	100
Loch Kirbister		12.5	14.5	99
Staunton Harold		15.	15.	100
Roadford		18.5	21.5	100

Table 5.2 *Comparison of D_{FSR} and D_{crit} for 10,000-year flood, including reservoir effects*

Name		D_{FSR} *h*	D_{crit} *h*	$Q_{10\,000}(D_{FSR})$ as % of $Q_{10\,000}(D_{crit})$
Loch Craisg	OG	5.75	6.75	100
Little Denny	OG	9.25	9.25	100
Loch Gleann	OG	11.25	19.25	96
Parkhill House		13.5	17.5 / 18.5	100
Leperstone	OG	8.75	7.75	100
Higher Naden		4.75	3.75	98
Lower Carriston	OG	8.5	7.5	100
Nanpantan	OG	7.5	6.5	100
Upper Neuadd	OG	7.25	4.25	94
Crafnant		6.75	4.75	98
Usk		13.75	9.25	99
Colt Crag	OG	15.	15.	100
Loch Kirbister		12.5	11.5	100
Staunton Harold		15.	13.	99
Roadford		16.5	13.5	99

Attenuation of design floods
It is also evident from values of the attenuation ratio, α_1, presented in Table 5.4, that reservoir effects are generally less pronounced in rarer events. This attenuation ratio is the ratio of the outflow peak to the inflow peak for the spillway design flood of the stated rarity. Only in exceptional cases does α_1 decrease with increasing return period. The discharge-head relationship assumed for Parkhill House is consistent with severe throttling of floods beyond its design standard.

For reservoirs which are small in comparison to their catchment area (e.g. Higher Naden, Nanpantan), the reservoir routing effect is generally weak. The index:

$$LAND = 1 - RESAREA/AREA \qquad 5.3$$

represents the fraction of the catchment that is covered by land (as opposed to the reservoir's lake) and partly indexes this behaviour. However, the reservoir routing effect is further modified if the spillweir length is unusually generous or unusually meagre. This is perhaps more simply thought of in terms of whether the design peak water level is unusually low or unusually high: 1.5 m above sill level is a typical value for a UK impounding reservoir.

Formulations
Included in Chapter 9 are approximate formulae which allow estimation of RLAG and α_1 prior to routing. These are tailored to cases where the main spillway is of a conventional 'ogee' type. However, given the ease with which reservoir routing can be performed in Micro-FSR, the formulae are instructive rather than essential.

Pre- and post-reservoir comparison
A rather different attenuation ratio is included in Table 5.5. This makes a direct comparison of design floods before and after reservoir construction. The attenuation effect is always rather weaker for the design floods (Table 5.4) than for the pre- and post-reservoir comparison (Table 5.5). This arises from the explicit allowance for reservoir lag made in the choice of design storm duration by Equation 5.1.

The flood protection afforded by impounding reservoirs may be illusory. Construction of a major reservoir is likely to be followed by a progressive diminution of watercourse capacity downstream as the river channel adjusts to lower flows and less frequent floods. The 'benefits' may be quickly absorbed as man exploits previously flood-prone riparian areas. Thus the effect may be to reduce the frequency, but to magnify the impact, of flooding downstream.

Table 5.3 *Reservoir lag, RLAG (h)*

Name		Return period, T years 10	100	1000	10 000	LAND index
Loch Craisg	OG	1.57	1.44	1.30	1.16	0.90
Little Denny	OG	2.77	2.59	2.39	2.17	0.88
Loch Gleann	OG	3.80	3.46	3.19	2.84	0.89
Parkhill House		3.15	3.09	3.67	4.45	0.98
Leperstone	OG	2.46	2.18	1.99	1.84	0.82
Higher Naden		0.70	0.59	0.48	0.36	0.99
Lower Carriston	OG	1.68	1.46	1.30	1.11	0.98
Nanpantan	OG	0.87	0.75	0.65	0.55	0.99
Upper Neuadd	OG	1.19	1.08	0.97	0.86	0.96
Crafnant		3.01	1.79	1.23	0.86	0.97
Usk		3.97	3.54	3.07	2.77	0.91
Colt Crag	OG	3.70	3.38	3.06	2.60	0.95
Loch Kirbister		2.17	2.07	2.04	2.05	0.95
Staunton Harold		3.41	3.15	2.83	2.40	0.97
Roadford		5.10	4.83	4.52	3.91	0.91

Table 5.4 *Attenuation ratio,* α_1

Name		Return period, T years				LAND
		10	100	1000	10000	index
Loch Craisg	OG	0.60	0.65	0.68	0.73	0.90
Little Denny	OG	0.49	0.53	0.56	0.60	0.88
Loch Gleann	OG	0.39	0.39	0.41	0.43	0.89
Parkhill House		0.51	0.51	0.42	0.34	0.98
Leperstone	OG	0.48	0.51	0.54	0.59	0.82
Higher Naden		0.89	0.91	0.94	0.97	0.99
Lower Carriston	OG	0.84	0.86	0.89	0.92	0.98
Nanpantan	OG	0.95	0.97	0.97	0.98	0.99
Upper Neuadd	OG	0.74	0.77	0.80	0.84	0.96
Crafnant		0.44	0.61	0.76	0.86	0.97
Usk		0.47	0.50	0.55	0.59	0.91
Colt Crag	OG	0.77	0.80	0.84	0.86	0.95
Loch Kirbister		0.83	0.84	0.84	0.84	0.95
Staunton Harold		0.77	0.80	0.83	0.88	0.97
Roadford		0.49	0.49	0.53	0.58	0.91

Table 5.5 *Attenuation ratio,* α_2*, for pre- and post-reservoir design floods*

Name		Return period, T years				LAND
		10	100	1000	10000	index
Loch Craisg	OG	0.52	0.57	0.59	0.62	0.90
Little Denny	OG	0.38	0.40	0.42	0.45	0.88
Loch Gleann	OG	0.30	0.30	0.30	0.31	0.89
Parkhill House		0.51	0.49	0.38	0.29	0.98
Leperstone	OG	0.38	0.40	0.42	0.45	0.82
Higher Naden		0.86	0.88	0.90	0.92	0.99
Lower Carriston	OG	0.82	0.84	0.86	0.88	0.98
Nanpantan	OG	0.95	0.95	0.95	0.96	0.99
Upper Neuadd	OG	0.67	0.67	0.69	0.69	0.96
Crafnant		0.37	0.52	0.64	0.72	0.97
Usk		0.37	0.38	0.40	0.41	0.91
Colt Crag	OG	0.73	0.75	0.77	0.79	0.95
Loch Kirbister		0.78	0.80	0.78	0.77	0.95
Staunton Harold		0.75	0.77	0.81	0.85	0.97
Roadford		0.39	0.39	0.40	0.45	0.91

Table 5.6 *Comparison of pre- and post-reservoir estimates of PMF*

Name		Pre-reservoir case			Post-reservoir case			α_3
		D_{FSR} *h*	PMF $m^3\ s^{-1}$	Season	D_{FSR} *h*	PMF $m^3\ s^{-1}$	Season	
Loch Craisg	OG	2.25	10.39	S	4.25	5.96	S	0.57
Little Denny	OG	3.25	12.50	S	6.25	4.85	S	0.39
Loch Gleann	OG	2.75	16.46	W	8.75	5.95	W	0.36
Parkhill House		10.5	8.43	S	13.5	1.59	W	0.19
Leperstone	OG	2.75	17.80	S	6.25	8.34	W	0.47
Higher Naden		2.75	75.61	S	3.25	69.61	S	0.92
Lower Carriston	OG	4.5	33.88	S	5.5	28.54	S	0.84
Nanpantan	OG	4.5	40.94	S	5.5	39.55	S	0.97
Upper Neuadd	OG	2.75	133.29	S	5.25	95.02	S	0.71
Crafnant		3.25	95.11	S	5.25	79.83	W	0.84
Usk		4.25	217.44	S	10.25	92.12	W	0.42
Colt Crag	OG	7.	127.19	S	11.	96.97	S	0.76
Loch Kirbister		5.5	133.25	S	9.5	93.31	W	0.70
Staunton Harold		7.	166.42	S	11.	128.59	S	0.77
Roadford		5.5	377.78	S	12.5	157.91	W	0.42

PMF comparison

Table 5.6 compares pre- and post-reservoir estimates of PMF, the former being taken from Table 4.2.

These results should be interpreted with caution: in particular cases, the discharge-head relationship assumed may not be valid for such an extreme event. However, a theme is the pattern of behaviour whereby catchments that are typically 'summer-critical' in the pre-reservoir condition can become 'winter-critical' in the post-reservoir state. This arises from the extension of design storm duration by reservoir lag (Equation 5.1). It can be expected that this behaviour will be accentuated for middle and lower reservoirs in 'cascades' (see Section 5.3 and FSSR 10).

Table 1 of Document A specifies that a higher concurrent windspeed is appropriate in conjunction with the winter PMF than with the summer PMF. Where no wavewall is provided but the dam is exposed to wave attack, it is possible for the windspeed criterion to tip the balance from an otherwise 'summer-critical' case to 'winter-critical'.

PMF v T-year flood

Tables 5.7 and 5.8 conclude the PMF to T-year flood comparison begun in Section 4.4. Table 5.7 differs from Table 4.2 in that reservoir effects are now included.

This again supports the broad equivalences set down in Table 1 of Document A. However, the Table 5.7 comparison is approximate in that the alternative standards are not thus phrased: for example, 0.2 times the routed PMF is not quite the same as the routed 0.2 PMF. Hence it is Table 5.8 that provides the more definitive comparison.

On the evidence of Table 5.8 there is some tendency for the use of 0.2PMF to produce a more conservative design than Q_{150}, and for 0.3PMF to be more conservative than Q_{1000}. Pleasingly, the $Q_{10,000}$ and 0.5PMF alternatives appear to be well balanced.

A singular feature of Table 5.8 is that the Loch Gleann Q_{150} and Q_{1000} routed floods are so much lower than their factored PMF counterparts. In Section 4.3 it was noted that Loch Gleann is the only case study catchment for which the PMF is

Table 5.7 *Ratio of T-year flood to PMF, after reservoir routing*

Name	Q_{150} : PMF	Q_{1000} : PMF	Q_{10000} : PMF
Loch Craisg	0.19	0.29	0.54
Little Denny	0.20	0.30	0.55
Loch Gleann	0.14	0.20	0.36
Parkhill House	0.43	0.52	0.70
Leperstone	0.18	0.27	0.49
Higher Naden	0.19	0.30	0.54
Lower Carriston	0.18	0.27	0.46
Nanpantan	0.16	0.24	0.44
Upper Neuadd	0.22	0.33	0.57
Crafnant	0.17	0.29	0.60
Usk	0.19	0.30	0.54
Colt Crag	0.16	0.25	0.45
Loch Kirbister	0.19	0.28	0.47
Staunton Harold	0.15	0.23	0.43
Roadford	0.17	0.25	0.49
geometric mean:	0.19	0.28	0.50
nominal ratio:	0.20	0.30	0.50

a winter event in the unreservoired case (see Table 4.1). This effect is thought to stem from the moderately permeable nature of the catchment, so that the frozen ground assumption causes the winter case to dominate. In contrast, two of the case study catchments for which the Q_{150}, Q_{1000}, and $Q_{10,000}$ floods are notably higher than their factored PMF counterparts are high SAAR catchments in Wales.

The final two rows of Table 5.8 relate to the guidance given in Table 1 of Document A to "take the larger" of the two alternatives. Adherence to this will, of course, tend to lead to rather higher design floods than if one or other alternative is applied consistently.

5.3 Practical matters

Reservoir flood estimation is rarely straightforward, even with strict adherence to the flood standards given in Document A and exploitation of the extensive computational facilities of version 2 of the Micro-FSR package. For example, considerable thought must be given to establishing that the discharge-head characteristics of the reservoir are adequately known: they should take account of any limitations imposed by the approach to, or egress from, the spillweir itself. Another key concern is the use of local data, already referred to in Section 2.4.

There are often features which are idiosyncratic to the particular reservoir being considered. Diversions into or out of the natural catchment area provide one example. Not all possibilities are explicitly dealt with by the Micro-FSR package, but the experienced user can, with sleight of hand, represent most configurations.

Cascades

Flood estimation for reservoirs 'in cascade' is especially complicated. While FSSR 10 is a help, their treatment is rarely routine. The Micro-FSR package does not automate computation of the weighted reservoir lag, MRLAG, but it does permit retrieval of previously calculated hyetographs and hydrographs.

In practice it is worth exploiting the package to consider a range of design storm durations. *Golden Rule 1* is that floods from separate

Table 5.8 *Ratio of T-year flood to relevant fraction of PMF, after reservoir routing*

Name	Q_{150} : 0.2PMF	Q_{1000} : 0.3PMF	Q_{10000} : 0.5PMF
Loch Craisg	0.83	0.92	1.09
Little Denny	0.81	0.91	1.08
Loch Gleann	0.57	0.63	0.72
Parkhill House	1.00	1.00	1.02
Leperstone	0.87	0.92	1.01
Higher Naden	1.06	1.06	1.11
Lower Carriston	0.94	0.92	0.95
Nanpantan	0.84	0.83	0.90
Upper Neuadd	1.14	1.14	1.18
Crafnant	1.32	1.26	1.34
Usk	0.93	0.97	1.05
Colt Crag	0.80	0.84	0.93
Loch Kirbister	0.81	0.84	0.90
Staunton Harold	0.77	0.81	0.88
Roadford	0.86	0.92	1.06
geometric mean:	0.89	0.92	1.00
nominal ratio:	1.00	1.00	1.00
geometric mean ratio of larger to 1st alternative	1.16	1.12	1.05
geometric mean ratio of larger to 2nd alternative	1.03	1.03	1.06

tributaries should only be combined when they have been derived from the *same* design rainfall hyetograph. (It seems that this rule is not universally well understood. For example, in a river engineering application at the confluence of the Blackwater, the Calmore Canal and the River Test, Bircumshaw & Fenn (1991) combine design flows from the three tributaries that are not consistent with any particular storm.) Having simulated the system response to design storms of differing durations, it is possible to inspect the various reservoir lags and to determine which case best fulfils the duration criterion given in FSSR 10. Of course an alternative would be simply to adopt the storm duration that yields the worst case.

Golden Rule 2 is that separate analyses should be carried out to check each reservoir in the cascade. The climate characteristics, and the point-areal rainfall reduction factor (ARF), should be derived for the entire catchment to the reservoir being checked. With experience, the user can build up a number of uniquely labelled catchment files within Micro-FSR to record fully the calculations undertaken.

Catchwaters

Because the carrying capacity of catchwater systems is usually fairly small in comparison to the design flood coming from the natural catchment, it is reasonable in most cases to apply the design rainfall hyetograph, calculated for the natural catchment, to the diverted catchment also. The hydrograph representing the contribution from the diverted catchment should, of course, be truncated to represent the limited carrying capacity of the catchwater.

6 Rainfall

6.1 Seasonal estimates

Context
As discussed in Section 4.3, seasonal estimates of rainfall are generally required when estimating the Probable Maximum Flood. Table 3 of Document A (see Section 2.1) provides ratios of summer and winter values of probable maximum rainfall to the all-year values. The latter are calculated using a depth-duration curve based on maps of maximum 2- and 24-hour rainfall (FSR II, Figures 4.1 and 4.2). The second edition of Document A (ICE, 1989) confirms that interpolation between 2 and 24 hours should be undertaken on linear-log rather than log-log graph paper. In addition, a misprint in a column heading to the table of seasonal factors has been corrected. Partly because users had expressed some confusion about their basis, and partly to exploit the longer data records now available, it was decided that the present study should include an explicit review of the seasonal factors for durations of one to eight days.

The FSR method
The seasonal factors for probable maximum rainfall given in Table 3 of Document A are based on estimates of seasonal factors for 100-year rainfall depths in the UK. These were calculated in the FSR method (Volume II) by a two-stage process: Table 2.11 presents seasonal factors for 5-year rainfall depths and Table 3.9 indicates how 100-year factors are expected to relate to these. Unfortunately some minor inconsistencies occurred when compiling Table 3 of Document A. Correction of these is shown in Table 6.1 under the label "corrected values derived from FSR II", but it is seen that the largest discrepancy is only two percentage points.

Analysis
Following the classification used in the FSR, the available raingauge data were divided according to the gauged (or estimated) value of the long-term average annual rainfall, SAAR. For each gauge and each duration (1, 2, 4 and 8 days) the annual, summer and winter maxima were determined for each year of record. The seasons are as defined in the FSR: summer is May to October, winter is November to April. Following the approach of Dales & Reed (1989), maxima from different gauges were standardised by dividing by the mean annual maximum value, RBAR; only long-term records were used, ensuring that the mean was well defined by the data.

The method of probability weighted moments (Hosking *et al.*, 1984) was used to determine the parameters u, a and k of the generalised extreme value distribution of the standardised rainfall frequency, for annual, summer and winter maxima in turn. These parameters were

Table 6.1 *Seasonal variation in probable maximum rainfall*

SAAR (mm)	Number of gauges	Average length of record (years)	Seasonal PMP as percentage of all year value							
			Rainfall duration 1 day		Rainfall duration 2 days		Rainfall duration 4 days		Rainfall duration 8 days	
			Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter
500-600	27	59	100 *100*	56 (57) *55*	100 *100*	65 *63*	100 *100*	68 *64*	100 *100*	70 *67*
600-800	191	58	100 *100*	63 (64) *62*	100 *100*	69 (70) *69*	100 *100*	77 (79) *73*	100 *100*	82 (81) *80*
800-1000	83	58	100 *100*	72 (73) *70*	100 *100*	80 (81) *78*	100 *100*	84 (85) *84*	100 *100*	91 (90) *91*
1000-1400	72	56	100 *100*	84 (85) *79*	100 *100*	88 (89) *85*	100 *100*	92 (93) 92	100 *100*	97 *96*
1400-2000	22	53	100 *100*	99 (100) *99*	97 *90*	100 *100*	95 (97) 92	100 *100*	92 (94) *89*	100 *100*
> 2000	10	54	97 *92*	100 *100*	90 (91) *84*	100 *100*	91 (94) *88*	100 *100*	89 (91) *83*	100 *100*

100 From ICE guide (Table 3)
(99) Corrected values derived from FSR II
100 UK results

calculated from probability weighted moments aggregated for all gauges in the SAAR class, weighted by record length.

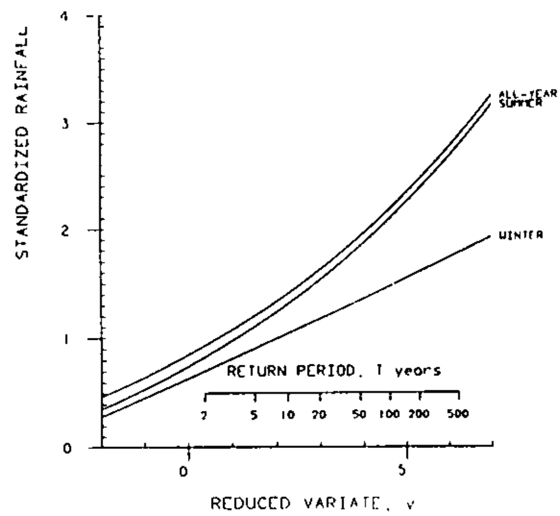
Examples of the rainfall frequency curves obtained are given in Figure 6.1. In each example, each of the three growth curves is standardised by the mean *annual* maximum value of the relevant duration. For 1-day rainfall at a 800-1000 mm SAAR site, the summer maxima are close to the all-year values, whereas the winter values are smaller, especially at the higher return periods. For 2-day rainfall at a 1000-1400 mm SAAR site (Figure 6.1b), the summer maxima are still higher than the winter maxima but less close to the all-year values. In the third example (Figure 6.1c), it is seen that the winter maxima for 8-day rainfall at a 1400-2000 mm SAAR site outstrip the summer values at all return periods.

Comparison
From Tables 6.2 and 6.3 it is seen that the above analyses led to summer and winter factors that were very similar to those derived in the FSR (new results in bold). Differences are no more than two percentage points for estimates of the seasonal factors for 5-year (M5) rainfall depths (Table 6.2), and no more than six percentage points for 100-year (M100) rainfall depths (Table 6.3). Looking only at the latter, it is seen that the present analysis yields slightly lower factors for summer rainfall in upland areas (e.g. SAAR > 1400 mm) and for winter rainfall in lowland areas.

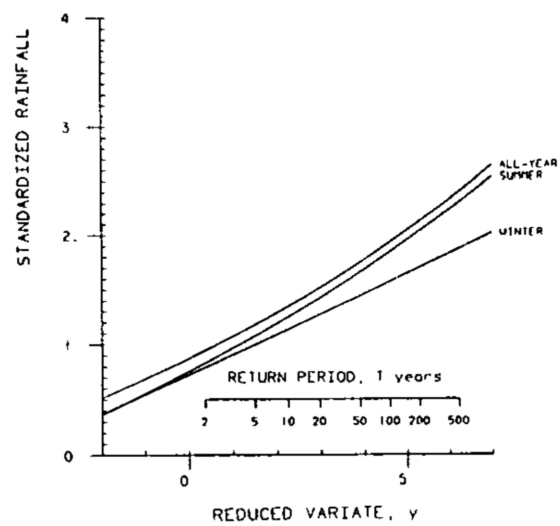
The results in Tables 6.2 and 6.3 stem from the analysis of 405 long-term gauges in the UK. The division between the SAAR classes is as indicated in the left-hand columns of Table 6.1. At an earlier stage, separate analyses were performed for gauges in England & Wales and Scotland & Northern Ireland, but national differences in seasonality of maximum rainfall were generally small and were judged insignificant, after taking sample size into account.

As part of a separate study, Dales & Reed (1989) derived rainfall growth curves for 1, 2, 4 and 8-day maximum rainfall partitioned by geographical region rather than by SAAR band. Chapter 9 of their report indicates that the proportion of annual maximum events occurring in winter is higher in the west than in the east, although this is not necessarily inconsistent with the FSR analysis.

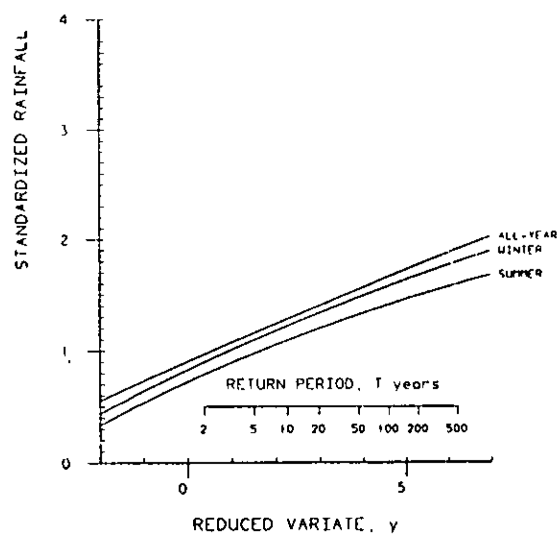
Implications
Implications for seasonal estimates of PMP can be deduced from Table 6.1, where the newly-derived factors are labelled "UK results". These derive from a minor transformation of the Table



a) *1-day rainfall SAAR: 800-1000 mm*



b) *2-day rainfall SAAR: 1000-1400 mm*



c) *8-day rainfall SAAR: 1400-2000 mm*

Figure 6.1 *Comparison of annual and seasonal rainfall growth curves*

Table 6.2 *Seasonal factors for M5 rainfall depths*

a) Summer as percentage of annual

SAAR mm		Duration 1 day		2 days		4 days		8 days	
500	600	97	**98**	96	**96**	96	**96**	95	**96**
600	800	96	**97**	95	**95**	95	**95**	93	**94**
800	1000	94	**92**	93	**92**	92	**92**	90	**90**
1000	1400	92	**92**	91	**91**	91	**91**	89	**90**
1400	2000	86	**87**	85	**86**	86	**87**	84	**84**
2000	plus	84	**87**	82	**85**	83	**87**	82	**84**

b) Winter as percentage of annual

SAAR mm		Duration 1 day		2 days		4 days		8 days	
500	600	65	**65**	71	**71**	73	**74**	74	**75**
600	800	70	**70**	74	**75**	79	**79**	81	**81**
800	1000	76	**75**	81	**81**	83	**85**	86	**87**
1000	1400	83	**82**	85	**85**	88	**87**	90	**90**
1400	2000	89	**89**	91	**91**	93	**93**	94	**95**
2000	plus	90	**90**	93	**93**	93	**93**	94	**94**

Table 6.3 *Seasonal factors for M100 rainfall depths*

a) Summer as percentage of annual

SAAR mm		Duration 1 day		2 days		4 days		8 days	
500	600	99	**100**	98	**100**	98	**99**	98	**99**
600	800	98	**100**	98	**99**	98	**99**	97	**97**
800	1000	98	**96**	97	**98**	97	**97**	96	**96**
1000	1400	97	**98**	96	**95**	96	**95**	95	**95**
1400	2000	91	**91**	90	**87**	91	**87**	89	**85**
2000	plus	89	**85**	86	**81**	88	**85**	86	**80**

b) Winter as percentage of annual

SAAR mm		Duration 1 day		2 days		4 days		8 days	
500	600	57	**55**	65	**63**	68	**63**	69	**66**
600	800	63	**62**	69	**68**	77	**72**	79	**78**
800	1000	72	**67**	79	**76**	82	**81**	86	**87**
1000	1400	82	**77**	85	**81**	89	**87**	92	**91**
1400	2000	91	**90**	93	**97**	94	**95**	95	**95**
2000	plus	92	**92**	94	**97**	94	**97**	95	**96**

6.3 factors: the season giving the higher factor is accorded a factor of 100 per cent (so that the all-year value of PMP is maintained), and the same uprating is applied to the factor for the other season (so that the seasonal ratio is maintained).

While the new analysis has resulted in slightly different seasonal factors, it is seen that in no case has there been a shift in the season yielding the all-year PMP. The most notable changes are the generally lower summer PMP values for upland areas.

The Institute holds few long-term records of hourly rainfall data and it was therefore impractical to verify the seasonal factors for durations of less than one day. In this sense the review of Table 3 of Document A is incomplete and the results are presented here for information only. The revisions have not been implemented within Version 2 of the Micro-FSR package and, accordingly, they did not figure in the case study estimates of PMF in Chapters 4 and 5.

6.2 Regional variations in rainfall frequency

Bootman & Willis (1981) demonstrated that the FSR method underestimated rainfall frequency in parts of Somerset. As well as confirming underestimates in the west country, Dales & Reed (1989) suggested that the FSR method may overestimate rainfall depths in north west England.

A partial revision (Stewart & Reed, 1989) has been achieved in south west England through the analysis of daily raingauge records. However, it is possible that discrepancies remain in the estimation of rainfall frequency for durations of only a few hours. Following the recognition by Dales & Reed (1989) that the FSR rainfall frequency estimation procedure is overgeneralised, a comprehensive reworking of UK rainfall frequency is required.

Temporal and spatial profiles for design storms in upland areas have been investigated in a companion study, funded by the DoE Reservoir Safety Research Commission, due to report soon.

6.3 Significance of heavy rainfall in historical UK dam safety incidents

Context

Although at present there is no central register of UK dam failures and incidents, Binnie & Partners (1986) presented lists of such events compiled during research for the Department of the Environment (DoE). Their lists built on earlier surveys, notably those of Moffat (1975), Hughes (1981) and Charles (1984). None of these surveys explicitly examined the significance of heavy rainfall in these dam safety incidents. In contrast, Acreman (1989a) reports some extreme floods that have occurred in the UK, but not in the context of reservoir safety.

In another reservoir safety research project commissioned by DoE, Dales & Reed (1989) examined, through an analysis of spatial dependence in extreme rainfall, the collective risk of a design storm exceedance occurring at one of a collection of dams. As part of their work it was possible to exploit daily rainfall data to review the extent to which rainfall has contributed to past dam failures and incidents.

Contemporary accounts of the more serious events usually exist, but sometimes these are no more than newspaper reports: authoritative engineering reports are less often available. Where incidents have occurred many decades ago, it is not always practicable to refer to the original sources. As a consequence, a shorthand description is formed of an incident which is handed down from author to author. In those cases where catastrophic failure occurred, there may be doubt about the sequence of events: e.g. did internal erosion cause settlement which led to overtopping or did overtopping lead to erosion?

The aim of the study carried out as part of the present project was to take a fresh look at the extent to which heavy rainfall may have contributed to the incidents and failures listed by Binnie & Partners (1986).

The rainfall data

The Regional Flood and Storm Hazard Assessment project (Dales & Reed, 1989) analysed long-term records of daily rainfall for some 400 stations in the UK. In addition, the Institute of Hydrology holds comprehensive records of daily rainfall since 1961. These records were augmented by reference to *British Rainfall* yearbooks: the earliest held at the Institute is for 1865.

Method of analysis

The events analysed were those failures and incidents for which Binnie & Partners (1986) gave a precise date. Reference was made to Hughes (1981) and to Ordnance Survey maps and gazetteers to confirm or establish the location of each dam. A number of flaws in the details supplied were corrected, most often with respect to grid references and duplicate entries; several incorrect transcriptions of incident dates were also noted. For a few dams, lying on relatively large catchments, a catchment centroid was estimated. Otherwise the grid reference of the dam was taken to be sufficiently representative in the subsequent search for rainfall data.

The raingauge numbering system used by the UK Meteorological Office is based on river basins. The first step was to note the hydrometric area in which the dam was sited and the adjacent hydrometric areas.

A computer program was written to extract the relevant rainfall data from the archive. Daily rainfall values for 15 days before each incident, and for 5 days after, were tabulated for gauges operating in the relevant hydrometric areas. These were ranked in order of nearness to the incident site. The rainfall on the day of the incident (R_0 mm) was noted for the nearest gauge.

An Antecedent Precipitation Index (API) was evaluated using 15 days' data from the nearest gauge. The API provides an index of the cumulative wetness immediately before the incident. The API (mm) was defined by:

$$API = k^{0.5}(R_1 + k\,R_2 + k^2\,R_3 + ... + k^{14}\,R_{15}) \qquad 6.1$$

where R_i denotes the rainfall depth on the *i*th day preceding the incident and k is the daily recession constant. A value of k=0.8 was adopted which corresponds to a recession half-life of 3.1 days. The choice of API formulation is inevitably rather arbitrary: the one used here provides a longer-term index than the 5-day API used in the FSR (which took k=0.5).

Results

Table 6.4 summarises specific rainfall information for 31 incident dates and 38 dams. Events prior to 1875 were not reviewed. The distance d from the dam to the nearest gauge is also given. For incidents where d exceeds about 10 km, it is quite likely that rainfall records exist for some closer site, which might be traced by reference to manuscript archives held at the Meteorological Office.

From this coarse analysis, it is seen that eight of the incident dates had heavy rainfall and a further five occurred when antecedent conditions were exceptionally wet. In other cases there is an inference from *British Rainfall* (BR) of heavy rainfall in the vicinity. The seasonality of incidents shows that eight of them occurred in July, and heavy rainfall definitely occurred in at least five of these.

Current research in the Joint Probability Studies for Reservoir Flood Safety project is exploring the definition of a trigger variable:

$$T = R_0 + w\,API \qquad 6.2$$

where w is a weight representing the relative importance of antecedent rainfall compared to event rainfall. Such a variable has been found helpful in explaining the incidence of landslides in New Zealand (Crozier & Eyles, 1980). As an example, Figure 6.2 displays the Table 6.4 data using a weight of 2/3.

Discussion

Rainfall for one recent incident was also investigated. This occurred at Lambielethan reservoir, near St Andrews, on 23 September 1985, in a very wet spell, with 36 mm on the rain-day of 22 September 1985 and an API value of 58 mm. The event would plot slightly above the T=50 line on Figure 6.2.

Although not exhaustive, the analysis presented suggests that the role of heavy rainfall in triggering dam safety incidents has been appreciable. While *British Rainfall* is a valuable reference, its thoroughness and format have shifted many times in the past century; it therefore provides only a partial record of heavy rainfall episodes. Additional rainfall data are available at the Meteorological Office, mainly in manuscript form. The total number of daily raingauges operating in the UK is substantial, as it was throughout the last century. It seems likely that searches of these additional rainfall archives would add further to the list of dam safety incidents where heavy rainfall occurred.

In this study, reference was made to relatively few contemporary accounts (other than those in *British Rainfall*). While heavy rainfall clearly contributed in part to a significant proportion of events, the evidence seen did not point to snowmelt having played a major role in these historical dam failures and incidents.

Of course, the presence of heavy antecedent and event rainfall does not necessarily imply that inadequate flood provision was the primary cause of an incident. The effect of heavy rainfall or local runoff on an earthen embankment may in some instances have been to trigger an incident attributable to a latent geotechnical defect. However, even in such cases, it would appear valuable for the concurrent rainfall conditions to be established.

Should a central archive of UK dam safety incidents be set up, it is recommended that the analysis above be extended and the significance

Table 6.4 *Rainfall information for reservoir safety incidents*

Dam	Incident date			R_0	API	d	Additional rainfall notes
Blakeney Brook	15	7	1875	0	15	25	
Nanpantan	20	7	1875	50	34	7	
Dovestone Clough	29	5	1881	0	5	30	
Braithwaite	12	12	1898	8	67	18	
Long Dam Embsay Moor	3	6	1908	1	9	20	BR: c. 100mm on day
Upper Cregon	8	9	1908	19	17	50	
Clydach Vale	11	3	1910	1	26	26	BR: "heavy" on day
Blaenycym	16	1	1912	23	42	20	BR: unclear
Balthayock	8	7	1916	13	23	46	BR: >75mm on day
Skelmorlie	18	4	1925	5	71	7	BR: 25% higher at site
Coedty Eigiau	4	11	1925	7	86	16 13	
Rainbow Forge	9	7	1926	6	5	9	
Deep Hayes	11	7	1927	51	15	1	BR: 51mm in 75min
Kepple Cove	29	10	1927	15	98	11	Carling: >70mm on day
Bilberry	29	5	1944	114	12	13	BR: 77mm in 2h at Rhodeswood
Tumbleton Lake	14	8	1946	3	36	18	BR: >75mm on 12th
Spott Lake Thorters Stobshiel	12	8	1948	120 59	51 40	29 23 26	BR; Baxter (1949)
Baynam Lake	17	7	1955	93	10	5	
Blackbrook	11	2	1957	0	16	0	
Blithfield	16	2	1962	1	9	2	
Auchendores	31	12	1967	4	20	1	
Birsemore Loch	8	4	1968	0	16	5	
Ashford Chew Magna Durleigh	10	7	1968	81 144 83	15 9 11	0 2 0	
Lluest Wen	23	12	1969	7	65	1	
Warmwithens Cocker Cobb	24	11	1970	0	40	1 0	
Glenogle	10	3	1975	0	22	0	
Combs	29	11	1976	0	12	3	BWB: storm/gale/cold
Bentley Priory Park	24	2	1977	22	22	1	
Bagnam Lake	17	7	1977	7	3	1	

(BR = British Rainfall; BWB = British Waterways Board)

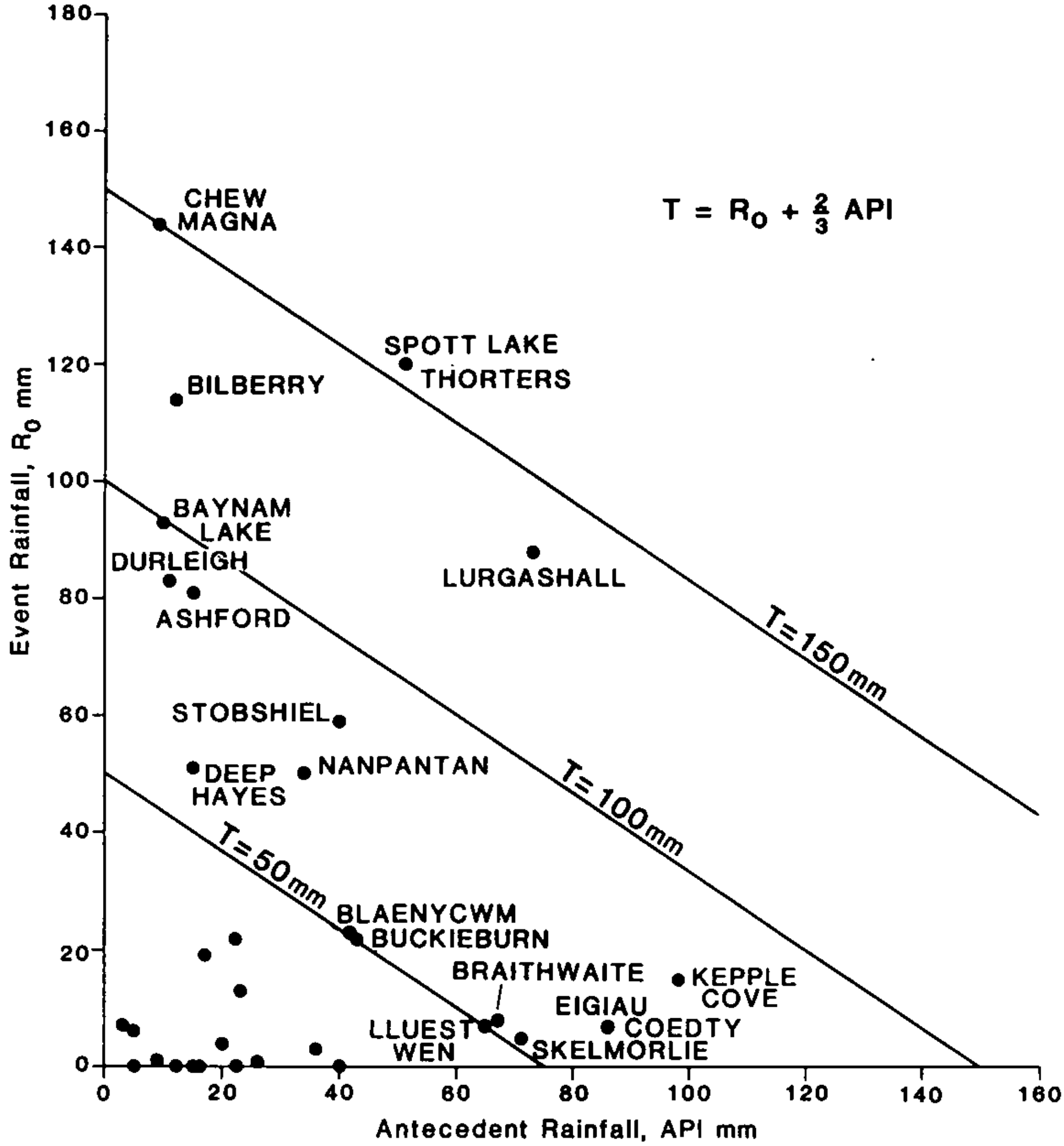


Figure 6.2 *Antecedent rainfall and event rainfall for UK dam safety incidents*

of heavy rainfall further elaborated. It might also be of interest to compile a list of extreme rainfall events which UK dams have successfully withstood without apparent incident.

6.4 Other aspects

Areal reduction factors (Stewart, 1989a) and temporal storm profiles (Srikanthan & McMahon, 1985; Stewart & Reynard, 1991) are the subjects of a separate research project commissioned by DoE. The reports are scheduled to appear in this series. Following the extreme storm in West Yorkshire on 19th May 1989 (Acreman, 1989b; Acreman & Collinge, 1991), the analysis of rainfall frequency continues to be of primary concern in reservoir flood estimation.

7 Snowmelt

7.1 Introduction

The occurrence of snow in Britain is highly irregular, in time and in space. A severe snowfall can be particularly damaging economically: major losses occur through transport dislocation and industry and commerce are now less resilient to the loss of electrical power, principally because of increased automation (Dombrowsky, 1990). While the Meteorological Office is keen to warn of heavy snowfalls and possible blizzard conditions, flood forecasting agencies such as the National Rivers Authority are concerned with forecasting the consequences of melting. A number of 'snowmelt models' are available to assist with this (e.g. Archer, 1983; Ferguson, 1984; WMO, 1986a; Harding & Moore, 1988), although experience in their use is limited by the infrequency of major snowmelt events in river systems for which flood warnings are required. An example of the occasional decisiveness of snowmelt in river flooding was the January 1982 flood on the Yorkshire Ouse (Rukin, 1982).

However, the importance of snowmelt in reservoir safety appraisals is rather different. The dichotomy in the reservoir flood standards table between use of a design flood based on PMF and one based on a T-year criterion does not ease discussion of the topic. An explicit allowance appears in the PMF procedure, but there has been concern, most notably voiced by Archer (1981), that this may be too small for use in some situations. To gain a clearer perspective it is helpful to consider first the statistical aspects of snowmelt occurrence.

7.2 T-year flood estimation — a joint probability problem

It is highly improbable in UK conditions that snowmelt alone would lead to exceedance of spillway capacity. This is because maximum snowmelt rates are considerably smaller than maximum rainfall rates for upland catchments in the UK climate. However there is legitimate concern that snowmelt could be important if it coincides with heavy rainfall and the rapid change in climatic conditions that would underlie such a scenario. There is also concern — based largely on impressions of the January 1809 and March 1947 floods in south and east England — that rapid snowmelt occurring while the ground beneath remains substantially frozen might be a scenario for an extreme flood on normally permeable catchments.

The statistical nature of the design against such conditions is immediately clear. One cannot assume that the 1000-year maximum reservoir outflow will be generated by, say, the 50-year rainfall coinciding with the 20-year snowmelt. From experienced gained in the study of other joint probability problems — most notably those of flood-tide and tide-surge interaction (Beran *et al.*, 1988; WMO, 1988; Tawn & Vassie, 1989) — there would appear to be difficulty in developing general rules. In almost every case an explicit model of the prototype system will be required if the frequency of exceedance of a given output condition is to be correctly established from knowledge of the frequency distributions of the inputs.

An exception is where the inputs being considered are of the same physical type, for example where both are rainfall or both are river flows. There has been substantial progress in generalising a model of spatial dependence in extreme rainfall (Dales & Reed, 1989). It may also be possible to develop procedures for estimating flood frequency beneath a major confluence from tributary flood data, without recourse to an explicit model of the intervening river system.

The snowmelt-rainfall problem is one of a number of joint probability problems being investigated in a new project for the DoE Reservoir Safety Research Commission. Another is the flood-wind problem that is relevant to specifying standards of wave protection at reservoir embankments.

The philosophy followed in the past has generally been to sidestep joint probability problems by assuming that one input factor is dominant. The design frequency of this variable is set equal to the required frequency of output, while the other input factors are set to some normal or typical condition. This practice may be adequate for some problems and can bring a certain consistency from one design to the next. However, simulation studies — to verify that such a 'package' *does* typically provide the desired standard of protection — are unavoidable if the method is to gain scientific approval.

The rainfall-runoff method of flood estimation given in the FSR uses this approach to determine the combination of design inputs from which to synthesise the T-year flood. Snowmelt is not one of these inputs: the method uses only a design rainstorm. However, the particular 'package' was verified by reference to frequency analyses of flood data for 81 catchments, and some of these floods were known to have a partial snowmelt origin.

7.3 Critique of snowmelt contribution to PMF

The FSR method of estimating the Probable Maximum Flood (PMF) includes an explicit snowmelt allowance which is optionally added to an estimated maximum rainfall. This allowance is a melt rate of 42 mm day^{-1} (1.75 mm h^{-1}) for as long as the 100-year maximum snow pack can sustain it. This allowance has come in for particular criticism, probably reflecting the way that it is specified, namely as a fixed rate irrespective of geographical location. The problem was partly defused by the introduction of seasonal calculations of PMF (ICE, 1978), since under this procedure many reservoired catchments are seen as 'summer-critical' (see Sections 4.3 and 5.2), but specification of a fixed snowmelt allowance irrespective of location remains something of a nonsense.

The earlier discussion of joint probability problems sheds some light here. The question arises: which is the more important input in terms of the snowmelt contribution to maximum floods, the water equivalent of the available snow pack or the melt rate?

As exemplified in Table 3.1, the median size of reservoired catchments in the UK is only a few km^2. Slopes are typically fairly steep so that catchment response times to heavy rainfall are generally short, chiefly in the range 1 to 12 h. In these circumstances it is fairly obvious that melt rate (rather than available water) will be the more important input. Had the snowmelt allowance in the PMF procedure been based on (say) a 50-year maximum melt rate, there might have been less controversy. The unacceptable feature of the FSR procedure is that the 1.75 mm h^{-1} rate represents a higher level of protection in those regions where snow (and hence snowmelt) is much less frequent than it does in snow prone areas such as north-east England.

This drawback was partly recognised at the time, and it was stated only that the 1.75 mm h^{-1} melt rate was a "suitably rare value for design purposes", i.e. for use in conjunction with an estimated maximum rainstorm to estimate PMF.

The origin of the rate is instructive. It was obtained from a degree-day formula:

$$\text{melt} = 12.1\,\rho \text{ mm day}^{-1}\,{}^\circ\text{C}^{-1} \qquad 7.1$$

where ρ is the density of lying snow. A value of $\rho = 0.4$ was assumed together with a 100-year maximum 3-hour air temperature *with snow lying*. The latter value of 8.6°C was derived from an extreme value analysis of 14 stations, each with about 15 years of record. This yielded:

$$\text{melt} = 12.1\,(0.4)\,8.6 = 41.6 \text{ mm day}^{-1} \textit{ or } 1.73 \text{ mm h}^{-1} \qquad 7.2$$

Whether this extreme satisfactorily represents a 100-year *conditional* melt rate at sites throughout the UK is open to conjecture. However, having adopted this generalisation, it was fundamental that the melt rate should have been moderated using Bayes theorem (of conditional probabilities) to reflect the known spatial variation in the frequency of snow lying. It is most surprising that this does not seem to have been remarked on previously.

Meteorologists have long garnered statistics of 'number of days with snow lying' (e.g. Manley, 1939; Dunsire, 1971; Hargreaves, 1976). Following publication of the FSR, Jackson produced an authoritative series of papers on UK snowfall (1977a), water equivalent (1977b), snow cover (1978a) and snowmelt (1978b), to which Folland *et al.* (1981) provide a summary. He was undeterred by data uncertainties. It is all too easy to find inconsistencies in precipitation, snow depth and water equivalent data and to conclude that most extreme observations are erroneous — as Schmidlin (1990) does! In particular, Jackson's generalisation of 'number of days of snow lying' could have provided a device for introducing a simple geographical component into the snowmelt allowance.

The snowmelt allowance is one example where preoccupation with the concept of a Probable Maximum Flood seems to have got in the way of a more workmanlike statistical approach. Simple return period criteria were used to determine the concurrent wind-induced wave surcharge allowance: why could not similar criteria have been applied to the snowmelt allowance? A logical approach would have been to use the same design windspeed in both the wave surcharge and snowmelt allowances, perhaps using a modified degree-day formulation for the latter.

7.4 A review of processes

The significance of the snowmelt contribution to catchment runoff is governed by the rate of melt runoff from the snow pack, the water equivalent of the pack, and the physical characteristics of the catchment. The melt runoff rate itself is determined by the rate of melt of the snow cover together with the characteristics of the snow pack.

Before melt

The precipitation form is related to air temperature. Above 3°C there is little doubt that this is chiefly rainfall; below 0°C it will be almost exclusively snow. According to the standard reference *Snow Hydrology* (USACE, 1956) 1.5°C is the median changeover temperature, but Norwegian references cited by Hartsveit (1984) report a figure of 1.1°C for the changeover.

Newly-fallen snow has low density, ranging from around 50 to 300 kg m^{-3}, the lowest values applying to 'dry' falls at very low air temperatures and the highest values being found for 'wet' or wind-packed snow. (The 'one foot equals one inch' rule corresponds to a density of 83 kg m^{-3} and will often underestimate the water equivalent, even of fresh snow.)

Once a pack has formed, the density increases with time as the snow crystals grow, typically moving into the range 300 to 600 kg m^{-3}. Although compaction, by wind and under its own weight, has some effect, the time scale of the density increase depends mainly on the air temperature and its changeability. Ferguson (1985) identifies repeated thaw-freeze cycles as a factor in producing very high densities in long-lived snow packs, having observed many core samples in the Cairngorm mountains with a density greater than 500 kg m^{-3}.

While recognising that densities greater than 400 kg m^{-3} could occur, Jackson (1977a) had assumed an average density of 150 kg m^{-3} in developing a map of 5-year maximum water equivalents. This assumption now appears to have little credibility in snow prone areas such as Scotland and north east England.

When major packs form, these are often as a result of several snowfalls. The different densities and layer characteristics present a particular bar to detailed modelling.

A wet snow pack at 0°C holds a certain amount of liquid water against gravity. This is called the free water of the pack, while its maximum content is known as the liquid water capacity. It is difficult to measure and there is little consensus about typical values. Hartsveit (1984) cites *Snow Hydrology* (USACE, 1956) as indicating 2 to 5 per cent by weight but Harding & Moore (1988) suggest that 10 per cent is typical. Of course, during rain and snowmelt episodes the percentage liquid content is raised further by water percolating through the pack.

A dense and grainy snow pack, which holds all the water it can hold against gravity, is said to be 'ripe'. Any rain or melt water entering the snow surface leads to a corresponding discharge of melt runoff from the pack. The percolation time varies with the depth and nature of the snow pack but is usually of the order of a few hours or less (Colbeck, 1972).

Before melt runoff can occur, the snow must reach melting point and must be given the latent heat of fusion. The heat deficit within the pack is referred to as the 'cold content' and is sometimes expressed as an equivalent depth of water.

Melt

The melting process requires energy which is mainly supplied as net radiation and sensible and latent heat transfers from the atmosphere. Heat flux from the ground is generally negligible but energy gained from rain is important when assessing the maximum melt rate that might accompany extreme rainfall. A formulation given by Hartsveit (1984) implies that 100 mm of rain at 8°C falling on a ripe snow pack might yield 10 mm of melt through this effect alone.

In Great Britain, snowmelt events are primarily associated with warm, moist, unstable air masses, characterised by total cloud cover and often accompanied by rainfall. Under these conditions the meteorological variables most relevant to heat transfer to the snow pack are air temperature, wind, net radiation, vapour pressure and precipitation.

Sensible and latent heat are exchanged with the atmosphere because of vertical gradients in air temperature and vapour pressure above the snow surface. Almost all of this heat is transferred by turbulence and these processes are therefore highly wind-dependent. The relative humidity of the air is also influential because some turbulent energy will be used for evaporation (rather than melting) if the atmosphere is unsaturated. Hartsveit (1984) concludes that the sensible and latent heat fluxes can be approximated by the product of a linear wind function and, respectively, the air temperature difference and the vapour pressure difference between the air and the snow surface.

The various radiation fluxes are complex but relatively well understood. Global radiation varies regularly with season and hour and irregularly with cloud cover. The albedo of the snow surface defines the proportion of the global radiation that is reflected. It is generally high but shows substantial variation with the characteristics of the snow cover: typically the albedo decays as the snow pack ages, so that more incoming radiation energy is retained in the pack.

While net radiation is dominant in the long-term fluctuation and decay of snow packs, its contribution to extreme melt rates is only moderately important. Few reliable data are available, but Hartsveit sketches an energy balance for a one-day melt of 110 mm on 21 May 1979 at Dyrdalsvatn, a site at 437 m altitude in maritime western Norway. The approximate contributions are sensible heat (60 per cent), latent heat (20 per cent) and net radiation (20 per cent); the main driving forces are a mean air temperature of 9°C and a mean windspeed of about 7.5 m s^{-1}. Concurrent rainfall was minor (9.6 mm) in this instance. The weather was mainly cloudy but not heavily overcast.

Kuusisto (1986) reviews snowmelt energy balances presented in 20 research papers. He concludes that:

- the radiation balance and turbulent exchange processes play a major role; the contributions of heat from precipitation or heat at the ground surface are small or negligible;
- the radiation balance and sensible heat exchange are almost always positive during snowmelt periods;
- both evaporation and condensation may prevail during snowmelt (thus the latent heat flux may be negative or positive);
- in forest environments the radiation balance is usually the most important energy component (suggesting that high melt rates occur less readily in forests);
- on cloudy or rainy days, turbulent heat transfer dominates;
- a very intense snowmelt usually also requires a large turbulent heat transfer.

The phenomenon by which a ripe snow pack sometimes 'collapses' in the early stages of melt, yielding runoff at a higher rate than the melting rate, is not well understood. This goes back to the uncertainty about the form and size of the liquid water capacity of a snow pack. Another strand in the complex web of snowmelt processes is the influence of the snow pack in determining the air temperature immediately above its surface (reported by Harding, 1986).

Catchment factors

The most obvious catchment feature of relevance to snow and snowmelt is elevation. While temperature typically decreases with altitude (a lapse rate of 1°C per 100 metre rise is often assumed), windspeed generally increases with altitude. Johnson & Simpson (1991) quote mean gradients of -0.90 °C and 0.64 m s^{-1} per 100 m rise. Applying these average values in a simple snowmelt formula such as:

$$M = (1.0 + 0.5\,v)\,T \qquad 7.3$$

where M is melt rate (mm day^{-1}), T is air temperature (°C) and v is windspeed (m s^{-1}) at 10 m height, suggests that altitudinal variations in snowmelt may not always be marked: the decrease in temperature with altitude is partly offset by higher windspeed. (Consider a severe melt occurring at 8°C and 8 m s^{-1}: this gives a melt rate of 40 mm day^{-1}. A 100 m rise yields 7.1°C and 8.64 m s^{-1}, giving a melt rate of 37.8 mm day^{-1}.)

Land cover is another factor thought to have a significant effect on snowmelt rates. For typical upland reservoired catchments, the greatest contrast would appear to be between open moorland and heavy forest. Heat transfer from the air to the snow surface depends, amongst other things, on the aerodynamic roughness of the terrain. However, such observations as have been made indicate that melt rates from forested catchments are generally less than from otherwise similar grassland catchments (Anderson, 1976). This is thought to be because the aggravating effect of greater aerodynamic roughness is outweighed by the shelter which the forest provides to the snow cover: lower windspeed means lower melt rate. Shelter does of course have a profound effect on the spatial distribution of snow: large variations in depth occur over horizontal scales of 10 to 1000 m.

Translating an understanding of point processes to a catchment scale is no easier for snowmelt than for other hydrological phenomena. Kobayashi (1985) examined snowmelt events on a small catchment in one of the coldest regions of Japan and found that subsurface runoff was dominant in the resultant hydrograph. Evidence for this is found in the 3 to 4°C temperature of streamwater, even when the stream is perfectly insulated (by snow cover) from heat sources above. Kobayashi's observations were for seasonal snowmelt events with a strong diurnal cycle and conclusions for typical UK snowmelt events are unclear. However, the research serves to illustrate that variable 'contributing area' concepts may apply to snowmelt as well as to rainfall-induced runoff.

7.5 Models and formulae

A detailed review of snowmelt modelling is not attempted here. The simpler empirical methods based on degree-day concepts and a wind term (e.g. Equation 7.3) are more readily generalised for use at any site. Interestingly, Zuzel & Cox (1975) suggest that a more powerful melt prediction formula might be based on vapour pressure, net radiation and wind, i.e. omitting temperature; their analysis was, however, based on data from a single 11-day study period. However, the strong role played by the energy exchanges in controlling melt makes a physically-based approach (e.g. Morris, 1983) that much more relevant to snowmelt modelling than to rainfall-runoff modelling.

An extensive international intercomparison of snowmelt models ended inconclusively (WMO, 1986a). Harding & Moore (1988) attempt to be more discriminating but conclude that — for flood forecasting applications at least — the choice between models is largely determined by the available input data.

Ferguson (1985) considers a simple conceptual snowmelt model with three parameters (a melt coefficient, a temperature lapse rate and a hydrograph recession constant) and two initial conditions (the snow cover as a proportion of the catchment and the snow pack water equivalent). The model uses only precipitation and air temperature data, there being no explicit allowance for wind effects. Unfortunately he concludes that the model does not cope well with major rain-on-snow events.

7.6 Appraisal of Archer's suggested 5 mm h^{-1} allowance

The principal stimulus to include a review of snowmelt in the terms of reference of the study arose out of the work of Archer (1981, 1983, 1984), which received considerable exposure among reservoir safety engineers. Of particular concern were the suggestions (Archer, 1984) that a snowmelt rate of 5 mm h^{-1} would be "appropriate in most parts of the country", that this rate has an approximate 50-year return period, and that it be incorporated (with minor modification) in lieu of the 1.75 mm h^{-1} allowance in the estimation of the Probable Maximum Flood.

Archer bases these recommendations largely on catchment runoff rates observed on a number of small upland catchments in north east England, which he attributes principally to snowmelt. However, rainfall was a known contributing factor in many of the events, as Ferguson (1984) comments.

Many of Archer's data were made available to this study. They were augmented by data from additional recording raingauges and automatic weather stations. Insufficient time and expertise were available to determine unequivocally the proportion of the maximum runoff rate that could be correctly attributed to snowmelt. Indeed that would have required the development of new methods of rainfall-snowmelt-runoff analysis.

There are many specialist points of which the hydrologist closest to the data is likely to have the best grasp. For example, was the quality of flow gauging affected by ambient temperatures close to freezing? Were precipitation records — in particular, tilting syphon raingauge records — similarly adversely affected? Possibly the most difficult aspects to judge are the spatial representativeness of the rainfall data and the altitudinal representativeness of the temperature data.

With these caveats, a re-analysis is presented here of six of Archer's events, primarily using flow data for the Langdon and Harwood catchments. For some of the events it was also possible to examine data from the Moorhouse and Alston catchments. Some catchment details are given in Table 3.5 and in Archer (1981). The layout of the catchments and measurement sites is shown in Figure 7.1.

EVENT 1: 2nd January 1976
Runoff rates were exceptionally high: gauged records indicated peaks exceeding 8 mm h^{-1} at Langdon and Harwood, and more than 4 mm h^{-1} at Alston. However the peaks were short-lived. The timing of the runoff is consistent both with an 11-hour rainfall, commencing at 10.00 on the 2nd, and with a snowmelt episode driven by a concurrent heave in air temperature.

Rainfall records indicate that about 30 mm fell in an 11-hour period, with the bulk falling in a 7-hour period. It would therefore appear unlikely that rainfall could explain more than half of the peak runoff rate. Corroboration that snowmelt did indeed contribute half and more of the peak rate is not strong: snow depth (and water equivalent) depletion data are inconclusive. Gauging error in adverse conditions cannot be ruled out, but it would be unreasonable to discount the exceptionally high flow records for two catchments in this way.

Appraisal: The data suggest that a high rate of melt runoff occurred for about 12 hours, perhaps with a peak contribution of 5 mm h^{-1}. Gales accompanied the thaw and this event

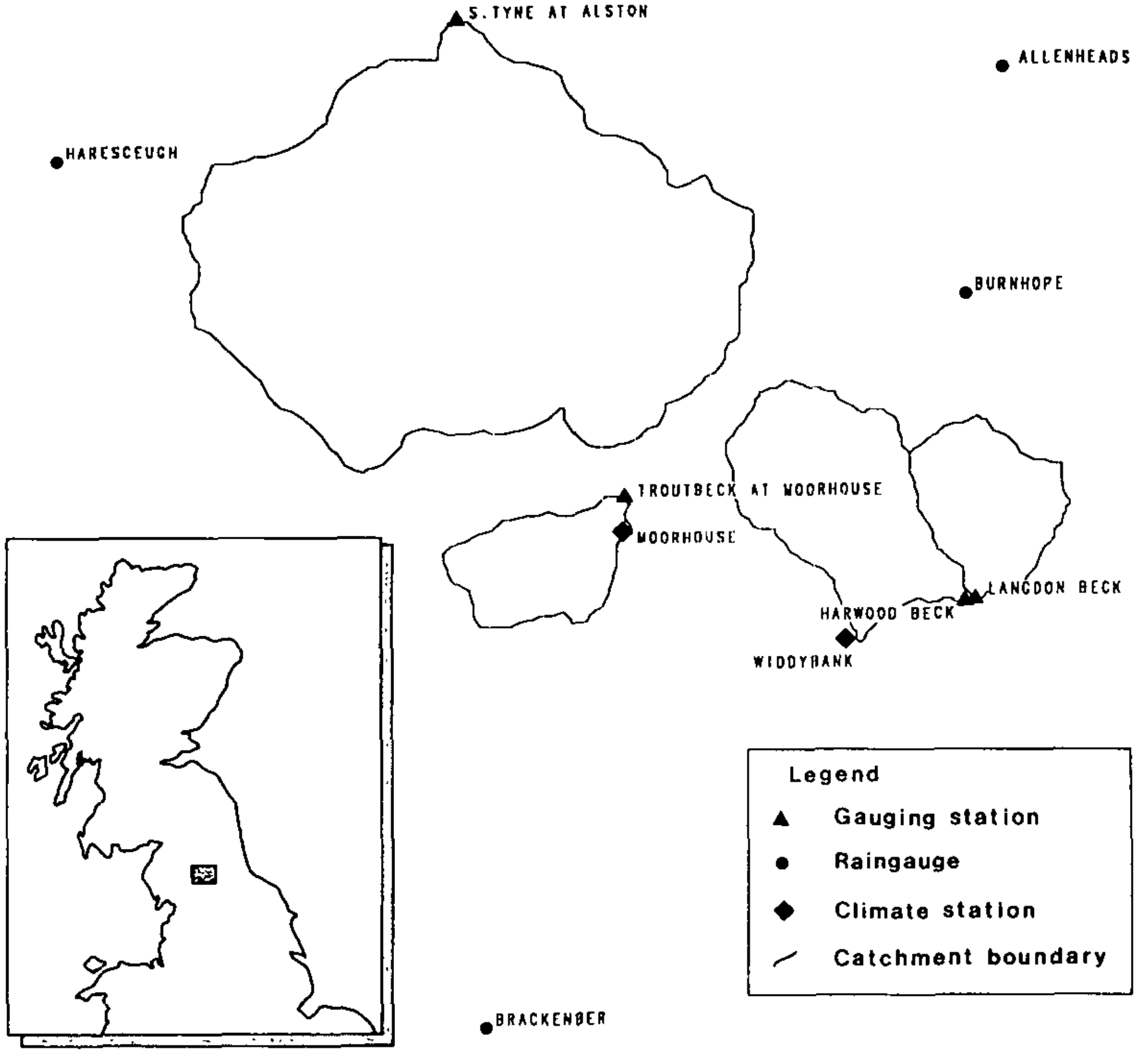


Figure 7.1 *Archer's snowmelt data: a location plan*

would appear to confirm the significance of high windspeeds in promoting rapid snowmelt.

EVENT 2: 24th February 1978
This was a complex runoff event persisting over several days. The main part of the event was between 09.00 on the 24th and 21.00 on the 25th. Peak runoff rates at Moorhouse and Langdon were close to 2.0 mm h^{-1}, while at Harwood a peak of 2.3 mm h^{-1} was attained. Runoff depths for the four-day period 23-26 February totalled about 100 mm on each of the catchments. Rainfall observations in the corresponding period averaged about 40 mm for six sites in the vicinity of the Harwood and Langdon catchments. However, the temporal pattern of rainfall conforms less well with the hydrographs than do air temperature data at Moorhouse, suggesting that this was primarily a snowmelt event. Data from further afield suggest that air temperatures remained at or above 5°C for the entire four-day period, with the warmest periods on the 24th. Observational notes at Widdybank confirm that a heavy thaw was in progress on 23 February.

Appraisal: Perhaps 75 per cent of the runoff originated from snowmelt. On this basis, a snowmelt contribution of 1.1 to 1.6 mm h^{-1} was sustained for a 24-hour period on the three catchments.

EVENT 3: 7th January 1979
The flow records for Langdon and Harwood show a smooth unimodal response. This appears to be consistent with a prior heave in air temperature although the Langdon hydrograph is particularly peaky, with about 40 per cent of the event runoff coming in only 6 hours. Peak runoff rates were about 2.7 mm h^{-1} at Langdon and 1.5 mm h^{-1} at Harwood. The flow record at Moorhouse appears inconsistent: it shows a prior runoff rate of 0.8 mm h^{-1} at a time when the Langdon and Harwood records show only a very small baseflow. The hydrograph is two-peaked, the earlier peak being absent in the Langdon and Harwood records.

The daily rainfall observations quoted by Archer do not tell the whole story. Tipping bucket records at Moorhouse, Allenheads and Burnhope reveal that there was heavier rainfall locally, all three gauges recording about 16 mm in the 16 hours preceding the hydrograph peaks. Given the degree of consistency between records, it would seem that rainfall was a significant factor in this event. Archer has maintained (personal communication, 1991) that the tipping bucket records are false, and that the consistency noted is due to snow melting into the gauges at the same time: the authors of this report do not agree with this interpretation.

Appraisal: Archer quotes a higher peak runoff rate for the Langdon station than is commensurate with the rating equation used in the re-analysis. Were it assumed that 75 per cent of the observed runoff derived from snowmelt, only at Moorhouse would this represent a melt rate greater than 1 mm h^{-1} sustained for more than 6 hours. However, there is doubt about the consistency of the Moorhouse flow record for this event, and also the clear impression that rainfall may have contributed rather more than 25 per cent of the runoff.

EVENT 4: 27th February 1979

Data for this event are illustrated in Figure 7.2. The hydrographs recorded at Alston and Harwood are very similar, showing a somewhat conical shape with a peak at about 20.00 h on the 27th. Both the Langdon and Moorhouse hydrographs are very different. While that at Langdon is broadly consistent with local rainfall (see below), the Moorhouse record is extremely odd, showing a peak runoff rate of about 3.8 mm h^{-1} sustained more or less uniformly from 20.00 on the 27th to 14.00 on the 28th.

Daily rainfall records are fairly sparse for the 26th-27th February. The Widdybank gauge had a two-day total of 8.1 mm, as quoted by Archer, divided evenly over the two days. But all other observations in the district were appreciably higher, ranging from 12.8 to 21.7 mm, with most of the rain falling on the 27th. Recording rain-gauge data from Haresceugh and Brackenber (see Figures 7.1 and 7.2) are broadly similar, and show a fall of 10 to 15 mm between about noon and 21.00 h, with a roughly triangular profile. The Widdybank observer reported rain turning to sleet at 12.30 h. However, possibly the most pertinent record is the tipping bucket data for Burnhope. These indicate that there was prolonged heavy rainfall, with 83.4 mm in 30 hours, only the last quarter of which was experienced at most gauges. From Figure 7.2 it is seen that there is a striking correspondence between the Burnhope hyetograph and the Langdon hydrographs, Langdon being the catchment closest to Burnhope.

Automatic weather station data for Moorhouse indicate that the air temperature rose above 0.6°C at about 09.00 h on the 26th and remained above 0.6°C until about 19.00 h on the 27th. The maximum temperature in this potential melt period was 1.9°C around 20.00 on the 26th, with a mean for the 34-hour period of 1.5°C. These data are broadly consistent with manually-read maximum and minimum temperatures taken at Moorhouse and at Widdybank. Concurrent windspeeds at Moorhouse were fairly high, exceeding 8.3 m s^{-1} from 08.00 to 19.00 h on the 27th, with a mean of 9.9 m s^{-1} for this 11-hour period. Snow depth data at Moorhouse indicate a depletion from 55 cm to 47 cm in the 24 hours ending at 09.00 h on the 27th; those at Widdybank indicate a depletion from 10 cm to 8 cm.

Of particular note is the pronounced frost throughout the 28th. The air temperature record provides no support for the extended period of high runoff recorded at Moorhouse.

Appraisal: The flow record for Moorhouse is grossly inconsistent, both with other stations in the area and with the primary forcing variables of rain, temperature and windspeed. The evidence points to a classical 'rain on snow' event, with the rainfall accounting for the larger part. The much higher runoff rates at Langdon than at Harwood or Alston would appear to be more readily attributable to differences in rainfall than to different conditions for snowmelt. Archer's interpretation of this event is very different. He asserts (personal communication, 1991) that the Burnhope tipping bucket rain recorder was acting as a melt gauge, but he was unable to offer an alternative explanation for the disparate runoff responses at Langdon and Harwood/Alston. We do not agree, and concluded that sustained snowmelt during this event was probably at about 1 mm h^{-1}.

Comment: It is regrettable that, in computer-ising daily raingauge data, the Meteorological Office has often chosen to omit *all* data for station-months when there are serious problems with some daily readings, rather than to enter part-months. Because of the severe frost in mid-month, this is the case for February 1979 rainfall records in the study area. As a consequence, the data readily available to the analyst are generally those from the more accessible sites at lower altitude; such rainfall data may not be representative in conditions of orographic enhancement. But for the availability of the tipping bucket data from Burnhope, Archer's classification of the 26th/27th February 1979

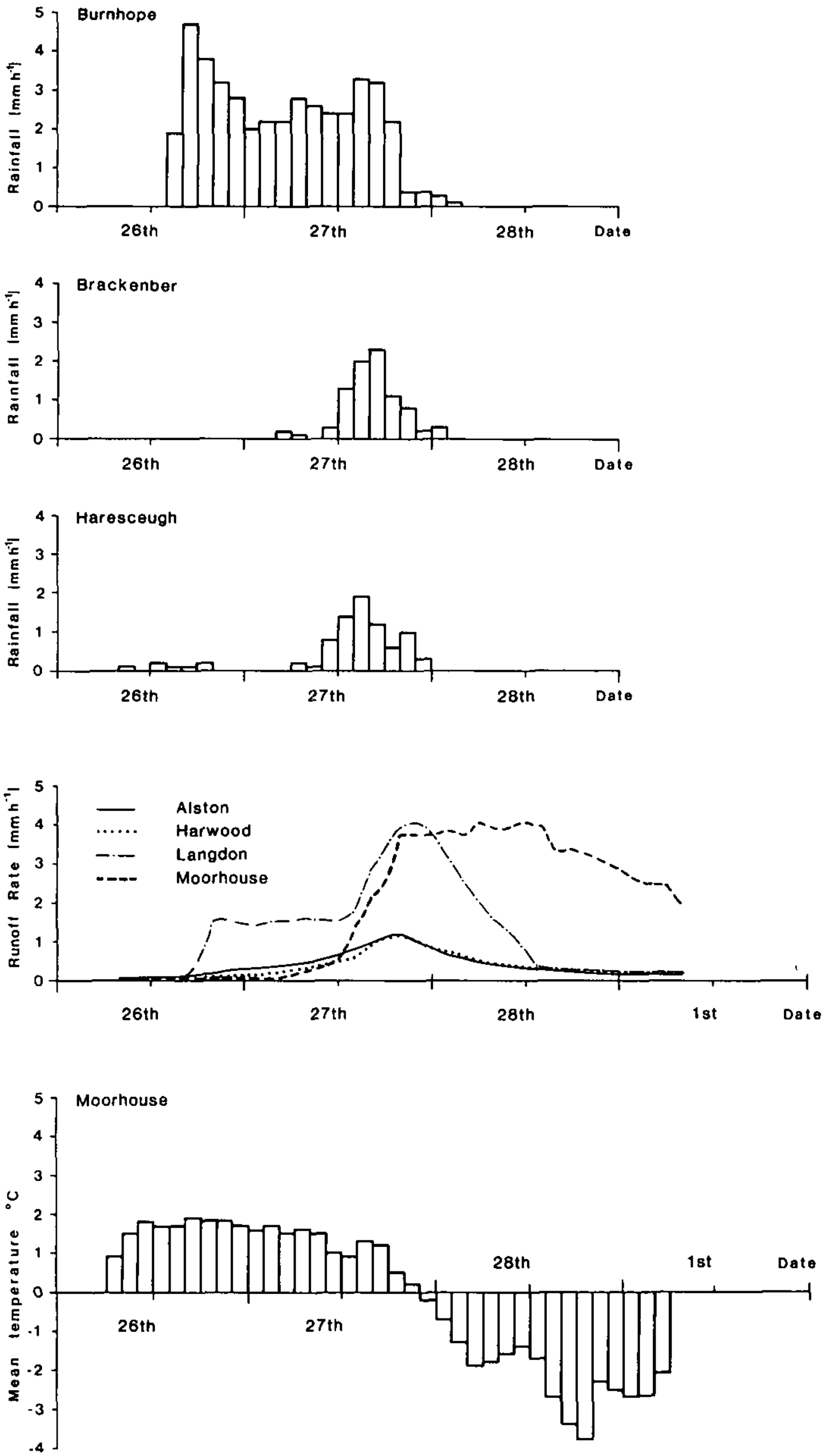


Figure 7.2 Rainfall, runoff and air temperature data, February/March 1979

event (as predominantly of snowmelt origin) would have been maintained.

EVENT 5: 2nd March 1979

This was a notable flood event and there is little doubt that snowmelt was a major factor. Peak runoff rates were about 2.5 mm h^{-1} at Alston, 4.1 mm h^{-1} at Harwood, 4.5 mm h^{-1} at Langdon and 7.2 mm h^{-1} at Moorhouse.

Air temperatures at Moorhouse exceeded 4°C from 03.00 on 2nd March until 09.00 on 3rd March, providing a fairly well defined period of sustained melt. The associated winds were high, with a mean speed of 9.9 m s^{-1} over this 30-hour period.

Rainfall was associated with both the start and the end of this main period of melt. Daily rainfall records indicate a two-day total of more than 30 mm. It appears that not all areas received heavy rainfall on 1st March but that those that did received depths of 10 to 23 mm. The tipping bucket gauge at Burnhope indicates that most of this fell in an 8-hour period beginning at 20.00 h on 1st March. Totals of 10 to 15 mm were widespread on the 2nd March rain-day, the recording raingauge data at Burnhope, Haresceugh and Brackenber indicating that most of this fell in a 10-hour period ending at 09.00 on 3rd March.

At Langdon and Harwood the hydrograph is characterised by a sudden rise above a plateau of about 2.1 mm h^{-1} which occurs at 20.00 h on 2nd March. Judging from the Burnhope record, this commences a couple of hours earlier than the second period of rainfall; the rise conforms rather better with the temporal pattern of windspeed which shows a marked increase from 13.00 to 22.00 h.

Appraisal: There is generally good consistency between the hydrograph records. Alston, Langdon and Harwood show very good agreement in runoff rates (mm h^{-1}) throughout the first half of the event. The fact that the second part of the event is less pronounced at Alston may reflect less uniform pack or melt conditions over the larger catchment. While doubts have been expressed above about the reliability of the Moorhouse record, it is not reasonable to discount it entirely in this event. The higher runoff rates could be partly explained by the likely higher windspeeds.

It is judged that rainfall contributed no more than about 15 per cent of the flood runoff, that a melt rate of 1.5 mm h^{-1} was sustained on all catchments for 30 hours, and that a melt rate of 3 mm h^{-1} was sustained on Harwood and Langdon for between 6 and 12 hours. There would appear to be some evidence that a melt rate of 4 mm h^{-1} was sustained for a full 24-hour period at Moorhouse.

EVENT 6: 25th March 1979

Temperature data for Moorhouse suggest a well-defined melt from 03.00 to 24.00 on 25th March. Flow records at Langdon, Harwood and Alston are all in general agreement, showing about 30 mm of event runoff with the bulk occurring in the 25th March rain-day (i.e. the 24-hour period ending at 09.00 on the 26th). The hydrographs are smooth and unimodal, the peak at Alston being appreciably higher (2.5 mm h^{-1}) than those at Harwood (1.7 mm h^{-1}) and at Langdon (1.4 mm h^{-1}).

Rainfall records for the 24th-25th March suggest a two-day depth of about 30 mm. Recording raingauge data at Burnhope, Haresceugh and Brackenber show somewhat different timings. Preferring the Burnhope and Brackenber data for transfer to Harwood and Langdon, it is inferred that 80 per cent of the two-day depth fell as rainfall in the 19-hour period 23.00 on the 24th to 18.00 on the 25th. The remainder fell as snow in the early hours of the 26th. Precipitation occurred about three hours earlier at Haresceugh than at Brackenber for this event. This may partly explain the rather earlier runoff response at Alston compared to Langdon and Harwood.

In interpreting the relative contributions of rainfall and snowmelt to the hydrograph, it is noteworthy that the hydrographs peaked three to six hours before the period of melt 'switched off', with the sharp drop in air temperature around 24.00 on the 25th. This is not explained by windspeed variations.

Appraisal: Perhaps 50 per cent of the runoff originated in snowmelt. A snowmelt contribution of 0.5 to 1.0 mm h^{-1} was sustained for a melt period of about 21 hours.

Summary

Archer sought to observe snowmelt runoff contributions at the catchment scale. Corresponding point melt rates can be expected to be higher. While the above analysis weakens some of Archer's supporting arguments, there is little doubt that melt rates as high as 5 mm h^{-1} *can* occur in UK conditions. In a detailed study using a weighing lysimeter at a site in maritime western Norway, Hartsveit (1984) observed a mean melt rate of 70 mm in 10 hours, and 110 mm in 24 hours, on 21 May 1979, with a concurrent rainfall of less than 10 mm in 24 hours. The observations were backed up by simulation of the snowmelt episode using climate data and an energy balance model. Using an energy budget approach, Mawdsley *et al.* (1991) derived estimates of probable maximum snowmelt in excess of 150 mm in 24 hours for catchments at 300 m altitude.

Thus the question is not whether sustained melts of 5 mm h^{-1} are feasible but, for a given location, whether they are at all likely. This returns the discussion to the basic weakness of the snowmelt allowance in the PMF procedure recommended in the FSR: it specifies a melt rate that is clearly much less frequent in one district (e.g. central southern England) than another (e.g. the Pennines).

7.7 T-year melt estimation — another joint probability problem

Development of a general procedure for estimating the T-year melt rate would be a complex undertaking. It is clear that at least three factors have a strong influence on melt rates: the initial presence of a substantial pack, a marked rise in temperature and high windspeeds. The 'Joint Probability Studies for Reservoir Flood Safety' project, recently commenced for the DoE Reservoir Safety Research Commission, will explore the dependence between these 'input' factors. However, because of the sparsity of climatological data in those areas where snowfalls are frequent, such a procedure may be difficult to generalise. A further complication is that in UK conditions the larger snowmelt events are generally accompanied by rainfall.

7.8 Historical aspects

Reservoir safety incidents have been listed by Hughes (1981), Binnie & Partners (1986) and others. While such lists are inevitably incomplete, snowmelt does not seem to have been a significant factor in those flood exceedances that have occurred. This contrasts starkly with river flooding records where snowmelt runoff has been strongly implicated in several very large events. The best known example is the March 1947 event which affected many major river basins in southern and eastern England. Other examples include floods in north east England in March 1963, those of December 1815, and the January 1809 floods — which are known to have affected much of southern and eastern England. Also memorable is the great Till flood of 16th January 1841 (Rodda *et al.*, 1976): a combination of lying snow and low antecedent temperature, followed by heavy rainfall and a rapid thaw, led to a classic extreme flood on a normally very permeable and unresponsive chalk catchment.

7.9 Snowmelt and reservoir flood safety: an operational approach

It may be possible to approach the problem operationally rather than to rely solely on high design standards. A large dense snow pack is a prerequisite to a major snowmelt contribution to a safety-threatening flood. When such conditions arise it is good practice to discharge water from the reservoir in advance of the melt. In the present climate, such large snow packs are relatively infrequent and explicit planning to modify reservoir operations is possibly less widespread than it was in snowier times such as the 1960s.

It is implied in the Floods and Reservoir Safety guide (ICE, 1978) that reservoir safety procedures ought not to rely on mechanisms which require human intervention or which may fail to operate in adverse conditions. While these principles are clear, their application is less so. Where is the line to be drawn between likely and unlikely, and between failsafe and risky? For example, following a very severe cold spell, ice jams *might* obstruct the safe discharge of flood water from an otherwise failsafe open spillway. Documented cases of ice floes on reservoirs are known in UK conditions (e.g. Whitson, 1955) but the possibility is not judged sufficiently likely to set explicit allowances.

The message is surely that vigilance is required at reservoirs whenever very unusual weather conditions occur. In extreme rainfall events, time-scales are generally too short to place any reliance on operational adjustments during the event. However, because a large snowmelt contribution cannot arise spontaneously, it is possible that the overall design for reservoir flood safety might augment explicit allowances for snow melt with some requirement for prudent operations.

8 Rainfall-runoff modelling

8.1 Review

A feature of hydrological research since publication of the Flood Studies Report in 1975 has been the development of statistically and/or physically-based rainfall-runoff models (e.g. Moore & Clarke, 1981; Beven & Wood, 1983; Abbott *et al.*, 1986; Calver, 1988; Jakeman *et al.*, 1990), some of them particularly tailored to small upland catchments with severe topography. Beven (1989) provides a critical review. For reasons largely anticipated by Lowing & Reed (1981), these developments have had little impact on rainfall-runoff methods of design flood estimation.

The aim of a rainfall-runoff model is to estimate the discharge response of a catchment to a rainfall event. The two extremes of rainfall-runoff modelling are the systems approach and the physics-based approach. In the systems approach an attempt is made to relate time series of input (rainfall) and output (discharge) data without any explicit representation of the physical processes involved. In contrast, a physics-based model uses physical laws to explain those processes occurring within a catchment. In between these two extremes lies the conceptual model, which uses empirical functions derived from a knowledge of input and output data to represent catchment processes in a simplified form.

The FSR rainfall-runoff method invokes a type of unit hydrograph model, which is entrenched in design practice for two principal reasons. Firstly, it is the only rainfall-runoff model which has been generally calibrated for use at any site in the UK. Secondly, its simple structure permits the incorporation of local data in a relatively straightforward manner. However, as Boorman *et al.* (1990) demonstrate, the FSR rainfall-runoff method of flood estimation is far from perfect when used in 'ungauged' form, i.e. where the model parameters are estimated from catchment and climate characteristics alone.

Developments directly relevant to the FSR rainfall-runoff method include: improved methods of deriving a catchment average unit hydrograph (Boorman & Reed, 1981), an improved method of calculating a catchment average rainfall profile (Jones, 1983), an enhancement to the S-curve method of transforming a unit hydrograph from site to site or from period to period (Reed, 1985), and construction of a comprehensive national flood event database (Houghton-Carr & Boorman, 1991). Urbanisation and agricultural/silvicultural drainage effects on flood generation, and how they may be accommodated within a rainfall-runoff method of flood estimation, have been considered respectively by Packman (1986) and Robinson (1990). There is as yet no objective definition of the margin at which heavily urbanised catchments are more appropriately treated by storm-sewer design methods. It is contended that land-use effects on flood runoff are *relatively* less significant when the design return period being considered is very rare. The thesis is that, in very extreme events, storm conditions (rather than ground conditions) are the dominant influence on the resulting flood. Reed (1987) describes some ways in which the FSR rainfall-runoff method can be applied to draw upon local knowledge, or non-standard data, in estimating design floods on otherwise ungauged catchments.

8.2 More detailed classification of soil types

Whereas catchment response times (such as the lag time between rainfall and runoff) can be validated by a relatively short period of instrumentation, it is much harder to refine estimates of standard percentage runoff (SPR) through gauging. The major part of inter-catchment variation in percentage runoff can be expected to reflect variation in soil types, so that more detailed classification and mapping of soils can be expected to be of particular value.

In the present method, SPR is generally estimated by reference to a five-class map of winter rainfall acceptance potential (WRAP), produced originally at 1:1,000,000 scale. In strategic research for the Ministry of Agriculture, Fisheries and Food, the Institute of Hydrology has collaborated with the Soil Survey and Land Research Centre (England & Wales) and the Macaulay Land Use Research Institute (Scotland) in production of a much more detailed hydrological classification of soil types (Boorman *et al.*, 1991). The general base for the Hydrology Of Soil Types (HOST) data set is soil mapping at 1:250,000 scale.

The HOST project is not yet complete. Soil maps at 1:250,000 scale are not yet available for Northern Ireland. New estimation equations for SPR are yet to be published. Arrangements for user access to the detailed gridded soil classifications still have to be resolved. However, it is already evident that the development will have significance for a very wide range of design flood estimation problems throughout the UK.

The FSR percentage runoff (PR) estimation equations, updated by Boorman (1985), are based on a data set comprising some 200 catchments. The same PR model is deemed applicable to all catchments. Clearly, if enough event data are available from a catchment, a local regression equation can be developed to replace the generalised percentage runoff model. In developing such a relationship, alternative independent variables — such as pre-event discharge — may prove efficacious. The process is now illustrated for the Croasdale Beck catchment. Of course, the benefit of attaining a better fit to gauged data for a particular catchment must be balanced against the risk that the resultant model may extrapolate idiosyncratically when applied to estimating very rare design floods.

8.3 An analysis of Croasdale Beck

Data from Croasdale Beck and Bottoms Beck were assembled in preparation for rainfall-runoff analysis, exploiting the good quality flow records available for these upland catchments, which form a pair close to Stocks reservoir in the Forest of Bowland in north west England. The Bottoms catchment is substantially forested, with largely clay soils and a rapid response to heavy rainfall, probably influenced by drainage. The plantations are mostly Sitka spruce and pine with some larch and deciduous trees. The Croasdale catchment has somewhat more permeable soils, with heather on upper slopes and bracken and grass on lower slopes and along the main stream courses (Wilson, 1970; Reece-Andrew, 1973). Some standard catchment characteristics are given in Table 3.5.

Within the budget of the project it was not possible to carry out detailed analyses of both catchments. A unit hydrograph model and a non-linear storage model (Reed, 1984) were applied to rainfall-runoff data for 20 flood events on Croasdale Beck, drawn from the period 1960-1969. In addition to its use in the FSR and follow-up work (Boorman, 1985), Croasdale Beck has previously been analysed by Mandeville — in development of the Isolated Event Model (NERC, 1975) — and by Reed (1976) in a unit hydrograph study. From this earlier work it was known that hourly data were too coarse to represent the flood response of the catchment comprehensively. Thus the runoff data were redigitised to yield data at 30-minute intervals.

A split-sample approach was initially adopted whereby rainfall-runoff models were fitted to a 'calibration' set of ten events and assessed on a separate 'test' set of ten events. It was found that the non-linear storage model yielded no significant improvement over the simpler variant for which the conceptual 'reservoir' has a storage-discharge equation with exponent (b) equal to 0.5; this corresponds to the Isolated Event Model (NERC, 1975). Fixed parameter values for the routing coefficient ($a = 0.21\ mm^{0.5}h^{0.5}$) and the pure time delay parameter ($d = 1.5$ h) provided generally good simulations of the flood events when the runoff coefficient (c) was permitted to vary from event to event, although the limited temporal resolution of the rainfall data was evident. Values of the runoff coefficient were approximately uniformly distributed between 0.5 and 0.97 with a mean value of 0.72. When all 20 events were considered, only two events yielded runoff coefficients outside this range: 0.35 for the 7th July 1964 event and 1.00 for the 29th September 1968 event.

The 20 events analysed correspond to all but four of the 24 events for which Boorman (1985) reports standard analyses by the FSR rainfall-runoff method: the events omitted were those on 5th July 1960, 18th December 1966, 1st and 2nd July 1968. Comparisons revealed that the runoff coefficients required in the Isolated Event Model were generally rather higher than the percentage runoff values observed in the unit hydrograph analysis (with a mean of 0.69 compared to a mean of 57 per cent). This is entirely to be expected because, despite its name, the Isolated Event Model simulates all runoff whereas the FSR unit hydrograph model simulates only rapid response runoff.

A fundamental requirement in both modelling approaches is to understand the variation of the runoff coefficient, or percentage runoff, from event to event. For Boorman's 24-event dataset, the FSR catchment wetness index (CWI mm) explains only 9 per cent of the variation in percentage runoff:

$$PR = 57.1 + 0.27\,(CWI - 125) \qquad 8.1$$

The logarithm of pre-event flow, ANSF (m^3s^{-1}), is a somewhat more powerful predictor, explaining 20 per cent of the variation in PR:

$$PR = 67.8 + 14.7\,\ln ANSF \qquad 8.2$$

These results can be contrasted with predictive equations for the runoff coefficient (c) of the Isolated Event Model developed on the 20-catchment data set. CWI explains only

4 per cent of the variation in c:

$$c = \{67.6 + 0.31\ (CWI - 125)\}/100 \qquad 8.3$$

whereas lnANSF explains 34 per cent of the variation, through the equation:

$$c = \{82.4 + 20.4\ \ln ANSF\}/100 \qquad 8.4$$

It has been found in other studies that the logarithm of antecedent flow is generally a better predictor of PR or c than is the FSR catchment wetness index. Whereas this is an important result in real-time forecasting applications of rainfall-runoff models (see Reed, 1984), this may not be the case in flood estimation applications, where a design value has to be found for the antecedent variable. It is, however, interesting that FSSR16 specifies design values of CWI and ANSF which are both based solely on the standard average annual rainfall, SAAR.

A second conclusion from the above is much more tentative. If antecedent flow is accepted as a meaningful and practical indicator, it would appear that inter-event variation in the runoff coefficient in the Isolated Event Model is more consistently related to antecedent condition than is percentage runoff in the unit hydrograph model. This may be in part a function of the different fitting procedures used in the two analyses. However, there is a hint that the structure of the Isolated Event Model is not without merit.

Were the Isolated Event Model to be applied more widely, it would seem likely that predictive equations for the runoff coefficient (c) and the pure time delay parameter (d) could be built up in much the same way as equations for the FSR rainfall-runoff percentage runoff (PR) and unit hydrograph time-to-peak parameter (Tp). The remaining parameter in the FSR rainfall-runoff method (ANSF) usually makes only a small contribution to the magnitude of resultant design floods. However, the remaining parameter in the Isolated Event Model, the routing coefficient (a), is an influential parameter whose estimation might be difficult to generalise. Mandeville fitted a version of the Isolated Event Model to 21 catchments but did not consider its generalisation for use at ungauged sites (see Section 7.3 of the Flood Studies Report, NERC, 1975).

While Equation 8.1 is statistically weak, it is encouraging to note that the multiplier of CWI is very similar to that in the generalised percentage runoff equation in the FSR rainfall-runoff method:

$$PR = SPR + 0.25\ (CWI - 125) \qquad 8.5$$

which applies for storm depths of 40mm or less (see FSSR16, Institute of Hydrology, or Boorman, 1985).

9 Reservoir routing

9.1 Introduction

The process of 'flood routing' is encountered in several practical problems. In situations such as closed sewer networks it is apparent that an explicitly hydraulic formulation is appropriate. The modelling of flood attenuation in rivers when the flow goes 'out of bank' is more contentious. Complex hydraulic models purport to cater for such conditions but their calibration appears at times to be no less subjective than much simpler methods. In comparison, the modelling of the passage of a flood through a reservoir is reasonably straightforward. Except for very special configurations, the passage is indifferent to hydraulic conditions at the inlet or approach conditions at the outlet. The moderating effect of the storage on an incoming flood can be represented by the geometrical relationship between storage and water level (the S-h relationship) and that by which the water level controls the discharge from the reservoir (the h-q relationship). This mathematical treatment is generally referred to as 'level-pool' flood routing.

The assumption of a 'level pool' is of course something of an approximation. Wind and seiche effects can produce pronounced differences. However, within present practice at least, it is customary to make an allowance for wind-induced wave run-up which is separate from that for the passage of a large flood. The interaction of flood runoff and wind set-up is one aspect being investigated in the recently-commenced "Joint Probability Studies for Reservoir Flood Safety" project.

Mathematically speaking, level-pool flood routing is capable of precise specification. Yet there is little recognition of this in standard engineering hydrology textbooks. Possibly because of a past necessity to offer graphical methods, little attention has been paid to the details of a numerical solution. The present report rectifies this by

- exposing the mathematics,
- presenting a solution method which caters for most reservoir configurations,
- giving detailed worked examples, and
- presenting the source code.

Much of this material is to be found in Appendix C: publication of the source code was an explicit part of the research contract. This will permit those who do not subscribe to the second version of Micro-FSR to share in this development, or at least to check their own computerised methods!

9.2 Definition of the routing problem

Let the volume of flood water stored in the reservoir at time t be S, defined in terms of water level, h, above a convenient datum. It is usually simplest if the sill of the lowest overflow is taken as the datum. (The flood water is, of course, stored only temporarily, in contrast to the long-term impoundment of water below the lowest overflow sill.) A flood arrives in two forms: as an inflow hydrograph (p) at the lake edge — representing flood runoff from the gathering grounds — and as direct rainfall (r) onto the lake surface. In some circumstances there may be extraneous imports from (or exports to) another catchment, but such cases can generally be dealt with by modifying the inflow hydrograph prior to flood routing (see Section 5.3).

The routing problem is to determine the resultant outflow hydrograph, q, and the water level graph, h, during passage of the flood. Of particular interest is the peak water level excluding wave effects, sometimes referred to as the 'flood rise' or the 'stillwater flood rise'.

In practice it is sufficient to set up a numerical scheme to obtain the water level graph. The discharge from the reservoir is then determined by exploiting the one-to-one relationship between discharge and water level. This is the h-q relationship, often referred to as the rating curve.

9.3 Formulation

Let the inflow p and outflow q be expressed in $m^3\,s^{-1}$, with water level h in m and storage S in m^3. To keep the formulation simple, the initial analysis takes the lake area A in m^2 and the rainfall rate r in $m\,s^{-1}$, although these are unfamiliar units for these variables.

The principle of conservation of mass yields the equation:

$$\frac{dS}{dt} = p + Ar - q \qquad 9.1$$

Noting that the area is simply the rate of change of storage with level, i.e.

$$A = \frac{dS}{dh} \qquad 9.2$$

Equation 9.1 can be written as:

$$A \frac{dh}{dt} = p + Ar - q \qquad 9.3$$

A preliminary to solving the routing problem is to eliminate A and q in favour of h using an 'area equation', A=A(h), and the 'rating equation', q=q(h), respectively.

Area-level relationship
The area-level equation represents the geometry of the lake and lake shore. Where the shore is steep it may be adequate to treat the reservoir as having a fixed area regardless of water level. The next simplest treatment is to consider that the lake area increases linearly with water level:

$$A = a_0 + a_1 h \qquad 9.4$$

and this is the formulation considered here. Only in exceptional cases will Equation 9.4 fail to represent the area variation adequately; one such example might be a 'manufactured' balancing pond where the shore gradient changed radically at some threshold level.

Some formulations of the reservoir routing problem prefer to work in terms of the storage-level (S-h) relationship rather than the area-level relationship. This does not seem very helpful. Firstly, if a storage-head relationship is available for an impounding reservoir, this will often have been developed to assist in monitoring the available stock when the reservoir is depleted. The table or equation may not extrapolate well to represent storage variations above sill level. In one example found, a cubic S-h equation — which provided a useful approximation to stock variations below sill level — led to storage estimates above sill level that were consistent with a *decrease* in area! It is intuitively easier to check that an A-h relationship makes sense. A second advantage of using the A-h formulation is that it simplifies the solution scheme, particularly when, as in Appendix C, explicit allowance is to be made for rain falling directly on the reservoir.

Discharge-level relationship: single equation
The 'rating equation' represents the various controls on discharge from the reservoir. In practice there may be more than one overflow weir and, in some instances, a piped or culverted discharge may also need to be represented. The solution procedure adopts the formulation:

$$q = bc\,(h - d)^e \quad \text{for} \quad h_{min} < h < h_{max} \qquad 9.5$$

where d is a datum level (corresponding to the zero flow level) and e is an exponent.

In many situations h_{min} will be equal to the datum level d and h_{max} will be unlimited (i.e. infinite). The exponent e is commonly 1.5 for open structures with crest control, such as a broad-crested weir or 'ogee' spillweir; for a drowned orifice it is 0.5. The parameter bc is a rating coefficient which, for a weir, would usually be the product of effective weir length (b) and a 'discharge coefficient' (c); a typical value of c is about 1.8 $m^{0.5}\,s^{-1}$. For a submerged orifice discharging freely, the parameter bc is the product of the cross-sectional area and another coefficient of discharge, for which a typical value is about 0.6 $m^{0.5}\,s^{-1}$; the water level is measured relative to the orifice centre. Flow behaviour in culverts is dependent on many factors and to represent discharge performance in detail it is necessary to refer to a hydraulic text such as French (1986).

The standard text Design of Small Dams (USDI, 1974) and the CIRIA guides to the design of flood storage ponds (Hall & Hockin, 1980; Hall *et al.*, 1991) give some guidance on outlet controls and their rating equations.

Discharge-level relationship: multiple equations
More generally, a *set* of equations is required to represent different behaviour in different water level ranges, or to represent more than one outlet device (e.g. a main spillweir and an auxiliary spillweir). The formulation builds these as a summation of several Equations 9.5:

$$q = \sum bc(h - d)^e \quad \text{for} \quad h_{min} < h < h_{max} \qquad 9.6$$

The formulation can be used to represent one or more outflow devices with multi-stage ratings by appropriate choices of h_{min} and h_{max}. The Micro-FSR package caters for either one or two explicit outlet devices but more complicated cases can be accommodated by sleight of hand.

Complex discharge ratings known only in the form of a rating table (i.e. discharges for set water levels, perhaps obtained by model testing of the outlet arrangements) — or set in that form for convenience — can also be represented. This flexibility is achieved by using a linear rating equation (i.e. with an exponent of unity) between each pair of (q,h) values. This is readily done by software which sets $h_{min} = h_1$, $h_{max} = h_2$, and fixes the remaining parameters (bc and d) such that $q = q_1$ at $h = h_1$ and $q = q_2$ at $h = h_2$. An example is included in Appendix C.

9.4 Solution

Insertion of Equations 9.6 and 9.4 into Equation 9.3 yields:

$$(a_0 + a_1 h) \frac{dh}{dt} = p + a_0 r - \sum bc(h - d)^e \qquad 9.7$$

if appropriate limits are retained on the terms in the summation. Given knowledge of the inflow hydrograph p, the rainfall rate r, and the initial water level, it is possible to solve Equation 9.7 for successive time-steps to obtain the water level graph during passage of the flood. The details of the numerical scheme are given in Appendix C2. It suffices here to note that an iterative solution, for which the Newton-Raphson method proves suitable, is generally required at each time-step.

A difficulty in the solution process arises when the water level at the end of a time-step is such that one or more terms in the summation become, or cease to be, active. This transition is tracked by checking that the water levels at the beginning and end of the time-step lie within the same 'range' of the q-h relationship; the check is made by reference to a table of all the levels at which the q-h relationship changes.

When such a condition is detected, a different numerical scheme is used to solve Equation 9.7. This is formulated *not* to seek the water level at the end of the standard time-step but to seek the *time* (within the time-step) at which h transcends the current range of the q-h relationship. (Iteration is not required in this solution scheme: the details are given in Appendix C3.) The standard solution scheme is then restarted — in the new water level range — from part-way through the time-step. Several transitions may occur within a single time-step when the water level is rising or falling rapidly.

9.5 Software

Listing of a non-interactive FORTRAN program for the reservoir routing problem is given in Appendix C5, with examples of its use preceding it in Appendix C4. Version 2 of the Micro-FSR package includes reservoir routing software based on that developed in the present study. The Micro-FSR implementation is, of course, fully interactive, with menus and graphical illustration of results.

9.6 Approximate formulae for reservoir lag and attenuation ratio

Reservoir lag time, RLAG, and the attenuation ratio, α_1, were defined in Section 5.2. While the availability of interactive software for reservoir routing in version 2 of Micro-FSR lessens the need for good estimates, some attention was given in the study to the derivation of approximate formulae for RLAG and α_1. There are several reasons why these might be of interest. Firstly, the delay and attenuation of the peak runoff rate are the properties that are most often referred to when considering a reservoir's impact on flood runoff. Secondly, the factors influencing these do not appear to be widely appreciated. Finally, 'back of the envelope' formulae still appeal to some users, particularly for the sizing of flood balancing ponds.

Reservoir response/lag time
Sælthun (1985) provides a formula for the 'response time', t_m, of a reservoir storage to a major flood. Converting the notation (but not the units) to those introduced in Section 9.3, the formula is:

$$t_m = 480\, A\, p_m^{-1/3}\, (bc)^{-2/3} \qquad 9.8$$

where A is the lake area at flood stage (km^2), b is the spillweir length (m), c is the discharge coefficient (m$^{0.5}$s^{-1}) and p_m is the mean inflow rate (m^{3}s^{-1}) over duration t_m. This response time, expressed in hours, is defined as the time taken for the reservoir outflow to rise to 80 per cent of a constant inflow rate. These definitions are not particularly convenient for practical use, since t_m is implicit in the definition of p_m, but the formula still has some merit.

Firstly, it reflects that the reservoir response time reduces as flood magnitude increases. Secondly, for spillweirs of 'ogee' type, the formula is dimensionally correct. The term $p_m^{2/3}\, (bc)^{-2/3}$ represents what the typical depth of flood storage above the spillweir would be if the flood passed through the reservoir unattenuated. Its product with lake area thus indexes the quantity of water temporarily stored. A mean residence time can then be defined by dividing this quantity by the mean inflow rate, p_m.

This suggests that an appropriate formula for the peak-to-peak reservoir lag might be:

$$RLAG = f\, A_0\, p_{max}^{-1/3}\, (bc)^{-2/3} \qquad 9.9$$

where p_{max} is the peak inflow rate and f is expected to be considerably smaller than 480 because the peak-to-peak lag time is much less than Sælthun's characteristic time. Here A_0 is the lake area at sill level, and the factor f is to be derived empirically. A satisfactory calibration was, however, hampered by limitations in the Equation 9.9 model and in the available data set.

Firstly, the 15 case study reservoirs being used were ungauged. Thus only hypothetical design floods could be routed. These were all of the standard FSR rainfall-runoff method type and it might be argued that these are not sufficiently representative of natural flood hydrographs. The chief concern is that, in addition to being strictly unimodal, the FSR design hydrographs have rather rounded (platykurtic) peaks. The second difficulty is that Equation 9.9 does not represent all the factors that influence reservoir lag times. Firstly, the increase of lake area above its sill level value is often appreciable, so that the reservoir area growth rate factor (A_1) ought to appear in the formulation. Secondly, the general temporal extent of the hydrograph and, specifically, its shape in the vicinity of the peak can be expected to have an appreciable effect on RLAG (and α_1). In the FSR rainfall-runoff method, the former can be represented by the unit hydrograph time-to-peak, Tp_0, but it is unclear how hydrograph shape at the peak is to be summarised, and where a more general data set might be found to calibrate a more sophisticated formula.

Such results as were obtained for Equation 9.9 suggest that a factor (f) of 120 is of the right order for design hydrographs typically produced by the FSR rainfall-runoff method. In contrast, the rather acute peak considered in Shaw's reservoir routing example (see Appendix C4.1) requires a factor of about 240.

Attenuation ratio, α_1

In the search for an approximate formula for the attenuation ratio, α_1, preference was again given to dimensional correctness. In keeping with Equations 9.8 and 9.9, a characteristic time was defined as:

$$RLAG_{syn} = 120\, A_0\, p_{max}^{-1/3}\, (bc)^{-2/3} \qquad 9.10$$

Because α_1 is itself dimensionless, a number of dimensionless factors were developed, including:

$$F_1 = RLAG_{syn}/Tp(0) \qquad 9.11$$

$$F_2 = A_0/AREA \qquad 9.12$$

and $$F_3 = (1 + SAAR/1000) \qquad 9.13$$

Note that 1000 mm is treated as a typical SAAR value in the definition of F_3. Tp(0) is the time-to-peak of an equivalent instantaneous unit hydrograph (see FSSR 16 and Boorman, 1985). These three factors index respectively: the temporal response characteristic of the reservoir in comparison to that of the catchment (F_1), the expanse of the reservoir lake in comparison to that of the catchment (F_2), and the degree of 'uplandishness' (F_3).

Two rather complicated equations were derived which jointly perform tolerably well for the particular combinations considered here (FSR design hydrographs and 'ogee' type spillweirs). The first equation:

$$\alpha_1 = 0.500\, F_1^{-0.61}\, F_2^{0.16}\, F_3^{0.51} \qquad 9.14$$
(factorial standard error = 1.07)

is *unsuitable* for lightly attenuated cases (say when $\alpha_1 > 0.8$) while

$$1 - \alpha_1 = 1.81\, F_1^{0.46}\, F_2^{0.59} \qquad 9.15$$
(factorial standard error = 1.21)

is *unsuitable* for heavily attenuated cases (say when $\alpha_1 < 0.3$). Either equation can be used for intermediate cases. The appearance of factor F_2 is not at all surprising and reflects the practice — evident, for example, in the original ICE Interim Report "Floods in relation to reservoir practice" — of assessing the likely significance of the reservoir routing effect by reference to the ratio of lake area to catchment area. In reality, the routing effect depends only on the details of the reservoir storage-discharge relationship and on the characteristics of the flood hydrograph, to which catchment area is merely a surrogate. Because some of the factors invoked are specific to the FSR rainfall-runoff method, it was impractical to provide an independent check of Equations 9.14 and 9.15.

Summary

An enterprising research student could doubtless develop improved formulae for RLAG and α_1, perhaps with wider applicability. These could be helpful in preliminary assessments of the reservoir routing effect. However, short-cuts would appear to be unnecessary, given the user-friendly software for reservoir routing available in version 2 of the Micro-FSR package.

10 Procedures and developments in other countries

10.1 Introduction

Procedural developments in reservoir flood estimation are discussed here for six countries: to have carried out a comprehensive review of research and development worldwide would have consumed all the resources available to the project. Perhaps the most notable omission is the Gradex method (Guillot & Duband, 1967; Guillot, 1973; Guillot, 1979), which is widely used to estimate extreme floods in France (e.g. Pinte & Rodriguez, 1991). The concluding Section refers more generally to research on reservoir risks.

10.2 Finland

The Finnish approach to dam safety is summarised by Loukola *et al.* (1985). New dam safety legislation came into force on 1 August 1984. The legislation applies to dams 3 m or more in height or which pose a particular hazard.

The Dam Safety Act recognises four categories of dam. Category P dams are those which in the case of an accident will endanger life or health, or cause serious damage to the environment or property. Category O dams are those which in the event of an accident will cause only minimal danger. Dams presenting an intermediate hazard are in category N, while category T dams are temporary structures. Minimum return periods are specified for the design of spillway capacities for new dams (see Table 10.1).

Table 10.1 *Spillway design floods: Finnish practice*

Category	Return period range
P	5000 to 10 000 years
N	500 to 1000 years
O	100 to 500 years

Shorter return periods may be considered adequate for temporary dams. Existing dams that do not meet these criteria must have their spillway capacity increased if, under such a flood, the dam would present a hazard to life or threaten major consequential damage (i.e. other than to the dam itself).

The basic method recommended for estimating the design flood is the extrapolation of a Gumbel distribution fitted to the annual maximum series of gauged floods. If fewer than 20 years of record are available, Loukola *et al.* (1985) recommend comparisons with nearby long-term stations or reference to a regional analysis of floods.

Many Finnish rivers are heavily regulated by lakes and reservoirs. Snowmelt floods are a regular occurrence in most years, April and May typically providing the annual maximum floods. However, average flood growth curves in Finland are no shallower in gradient than those in the UK (Gustard *et al.*, 1989). Thus the recommendation by Loukola *et al.* (1985) to base spillway design floods on simple extrapolation of peak flow records at the dam site is extraordinary. If followed, the guidance could lead to gross under- or over-estimation of design floods for a particular dam through over-reliance on statistically very short data series. It is regrettable that Kuusisto (1988) reiterated this advice with little qualification about the desirability of some form of regionalisation.

Loukola *et al.* (1985) state that appraisal techniques based on estimates of the Probable Maximum Flood were considered but rejected because of the relatively long discharge series available for most Finnish rivers. Presumably, PMF estimation would have been hampered by the significant role of snowmelt runoff in Finland. It is not clear how the peak flow estimate is converted to a hydrograph for the purpose of routing the design flood through the reservoir storage.

10.3 Sweden

Proposed spillway design flood standards for Sweden are described by Bergstrom *et al.* (1989). An unusual feature is the use of a unique 14-day design rainfall profile of unknown return period (possibly about 10,000 years). Corrections are made for geographical region, altitude, and catchment area. The basis of the latter is unclear but other publications (e.g. Vedin & Eriksson, 1988) suggest that the adjustment derives from a 'storm-centred' rather than a 'fixed' areal reduction factor. Areal reduction factors for design rainfalls in upland areas of the UK are examined by Stewart (in press).

10.4 Norway

The Norwegian approach to reservoir flood estimation is summarised by Sælthun & Andersen (1986). New regulations for dam design came into force on 1st January 1981. They apply to permanent dams more than 4 m in height or which impound more than 500,000 m^3.

Flood calculations are required for two key floods: the design flood and the Probable Maximum Flood (PMF). A 1000-year return period is specified for the design flood, which sets the standard for normal spillway operations; the PMF sets the standard for dam safety. The design flood is determined by some type of frequency analysis of peak flows. The PMF is calculated on the basis of Probable Maximum Precipitation (PMP) values and snowmelt estimates. Allowance is made for reservoir routing.

In general, spillway operation is designed so that flood discharges passed downstream are no greater than the natural condition for floods more frequent than the 1000-year event. The regulations recognise that, for rockfill dams, the effective design standard for the spillway is PMF, because rockfill dams cannot be allowed to overtop.

Many Norwegian reservoirs are constructed primarily for hydropower and catchment areas are typically very much greater than in the UK. Reservoir routing effects are often important and reservoirs are often critical to flood conditions building up over many days.

Estimation of 1000-year flood
The guidelines (Vassdragsdirektorat, 1986) recommend that several statistical distributions be considered when seeking an estimate of the 1000-year flood.

If more than 50 years of annual maximum flood data are available, the mean annual flood (QBAR) is estimated from the observed series, while the growth factor (Q1000/QBAR) is taken from a two- or three-parameter distribution fitted to the observed series. If only 30 to 50 years are available, a two-parameter distribution is to be used. If fewer than 30 years of data are available, the Q1000/QBAR growth factor is based on a regional analysis. If fewer than ten years of data are available, QBAR is estimated by correlation with other series in the region or by catchment characteristic formulae.

In many cases it is deemed appropriate to distinguish spring (largely snowmelt) and autumn (largely rainfall) floods. The spring floods yield a high QBAR and large hydrograph volume, but have only moderate growth rates. In contrast, the autumn floods stem from shorter duration events of high intensity to which steeper growth curves apply.

A rainfall-runoff approach to estimating the 1000-year flood is not generally recommended. This is because of the 'joint probability problem' of choosing appropriate initial catchment wetness and snowmelt/snow accumulations to combine with a 1000-year precipitation event to produce the required 1000-year flood.

Sælthun & Andersen (1986) describe what appears to be a fairly subjective method for converting statistically-derived estimates of the 1000-year peak instantaneous and/or peak 1-day flow into a design hydrograph suitable for reservoir routing. They caution against the practice of nesting 1000-year flows of different durations within a single design hydrograph.

Characteristic response time of reservoir
To assist in choosing an appropriate range of design flood durations for checking the spillway design, Sælthun (1985) provides a formula for the response time of a reservoir storage to a major flood. This was discussed in Section 9.6.

PMP
Guidance for estimating PMP is given by Forland (1984) and Forland & Kristoffersen (1989). Estimates based on Hershfield's method (Hershfield, 1961; WMO, 1986) are compared with those based on a diagram taken from the Flood Studies Report analysis (NERC, 1975, Figure II.2.4). This relates PMP to the M5 depth of the same duration and corresponds to an extrapolation of the FSR II MT/M5 rainfall growth factors to a return period of about 30,000 years. (The Flood Studies Report in fact preferred an alternative procedure for estimating PMP.) Forland presents a method for estimating M5 rainfall depths at any site in Norway. The comparison of the quasi-FSR estimates of one-day PMP with those derived by Hershfield's method indicated that the latter gave either higher or similar values.

Other parts of FSR II have also been borrowed: for example, the areal reduction factors that are used to convert point design rainfalls into equivalent catchment values. The justification for applying UK results to such a large extent is that the precipitation regimes are not very different and that the UK benefits from an exceptionally dense network of long-term raingauges. The Norwegian network is less extensive and the analysis of spatial variations in rainfall is inhibited by the extreme topography (Sælthun & Andersen, 1986).

It is understood that subsequent work at the Norwegian Meteorological Institute has sought to check the validity of the UK formulae by further analyses of Norwegian rainfall data. Special attention has been paid to deriving seasonal estimates of PMP.

Calculation of PMF
Seasonal influences on extreme floods are particularly marked in Norway. For some small catchments, the critical season is summer (high intensity storms). However, more generally it is late autumn, when combined rainfall and snowmelt floods can occur. On very large and/or heavily regulated catchments, the critical season is sometimes spring, where snowmelt dominates.

Snowmelt allowances are included in spring and autumn estimates of PMP. For autumn estimates, the temperature is set to the highest observed during rainfall after the normal date for snow cover in the catchment. Snowmelt is calculated as part of the hydrological model (see below) and uses altitude zones and empirical 'degree-day' factors. These range from 1.5 mm $°C^{-1}day^{-1}$ for dense forest to 3.5 mm $°C^{-1}day^{-1}$ for glaciers. Enhanced rates of melt are assumed for rain on snow, the degree-day factors being doubled.

The hydrological model recommended is a variant of the HBV model (Bergstrom, 1976), of which Lindstrom (1990) gives a description. As the name PQRUT intimates, the computer package incorporates allowance for reservoir and river routing (Andersen *et al.*, 1983).

Sælthun & Andersen (1986) recognise the anomalies that can occur in assessing design floods for a cascade of reservoirs. They indicate that a single design storm should be applied to the entire catchment to the reservoir under scrutiny.

10.5 Republic of South Africa

Recent floods
The Republic of South Africa (RSA) deserves mention because it has experienced a number of exceptional floods in recent years. The storm of 23rd-25th January 1981 affected about 50,000 km^2 of southern RSA. This was described at the time as the country's greatest natural catastrophe. The resultant floodwaters washed away much of the town of Laingsburg with the loss of 104 lives (Adamson, 1981).

While no major dam failed, the estimated peak discharge at Floriskraal Dam, just downstream of Laingsburg, was 77 per cent greater than the design flood, with spill taking place over the whole length of the concrete dam to a depth of 7.4 m above the spillway crest and 2.4 m above the top of the parapets. The effects were aggravated by a failure to open discharge gates.

A succession of extreme events occurred in the period March to May 1981, reflecting another feature of the region: that there is some persistence in extreme events. Tropical cyclone Domoina in January/February 1984 produced unprecedented flood peaks in northeast RSA which led to design exceedances at other dams (but no significant failures) and spectacular 'outliers' in a subsequent flood frequency analysis.

An even more serious series of floods occurred in September 1987 (Kovacs, 1988a) and February/March 1988. The former event is estimated to have cost 620 lives (Alexander, 1988). Design floods at several large dams were exceeded but the dams held; however, two moderately-sized dams, and some 400 smaller dams, were breached.

In discussing these events, Alexander (1988) recognises the difficulty of accommodating extreme events in a regional flood frequency analysis, where the meteorological basis of the outlying storms differ so much from that of other annual maximum events. In addition he draws attention to the difficulty of allowing for exceptionally wet antecedent conditions and he conjectures that such conditions are the norm rather than the exception when considering only the most extreme flood events.

Pegram & Adamson (1988) suggest that the former problem can be tackled by adopting a two-component extreme value representation (Arnell & Beran, 1987) for the distribution of annual maximum rainfalls.

Choice of PMF procedure
Alexander & Kovacs (1988) comment on the accumulated wisdom in South Africa that flood estimation methods based on maximum experienced floods in hydrologically similar regions (seen against the background of world maxima) are to be preferred to conventional statistical analyses or probable maximum flood estimates based on deterministic methods. This is a retreat from the part-envelope, part-deterministic approach favoured by Midgley *et al.* (1969). Alexander & Kovacs (1988) see a particular difficulty in maximum flood estimation in regions that are climatological transition zones, for example in the margin of those areas of northeast RSA that are recognised to be under the influence of tropical cyclones.

The Francou-Rodier method

The Department of Water Affairs preferred method of flood estimation for dam design is the Francou-Rodier empirical approach (Francou & Rodier, 1967). This is based on an envelope of maximum recorded flood peaks in a given region. The formula is:

$$QMAX/10^6 = (AREA/10^8)^{(1-0.1K)} \qquad 10.14$$

where QMAX is the estimated maximum (i.e. upper limit) flow in m^3s^{-1}, AREA is catchment area in km^2, and K is a regional coefficient. Kovacs (1980) proposed five maximum flood peak regions distinguished primarily by rainfall regime; each region has a different K value. Figure 10.1 is an updated compilation and is accompanied by a map of K for Africa south of latitude 18°S (Kovacs, 1988b).

The Francou-Rodier is more subtle than some other flood envelope methods — e.g. those based on the Normal Maximum Flood (once in widespread use in the UK). This is because the envelope for a given region is made to conform to a wider family of envelopes and, ultimately, to world maxima. Its proponents argue that the envelope is readily defined by gauged and historical data; Alexander (1988) suggests that palæoflood maxima might also be used.

That flood envelope methods drift upwards as more floods are observed, or come to light, is less of a damning criticism than at first appears. Scientifically more respectable methods based on quasi-physical estimates of PMP and rainfall-runoff modelling share this basic tendency. However, the failure of flood envelope methods to take explicit account of catchment factors (other than AREA) is seen by some as a sign of scientific bankruptcy (Beran, 1981).

Philosophy

The present preference for an envelope approach is not for want of philosophical questioning. Alexander (1988) and Kovacs & Alexander (1990) identify weaknesses in the rainfall-runoff and statistical alternatives.

Alexander suspects that rainfall-runoff methods of flood estimation are largely driven by the rainfall frequency relationship and neglect the important additional variability imposed by catchment processes — particularly antecedent

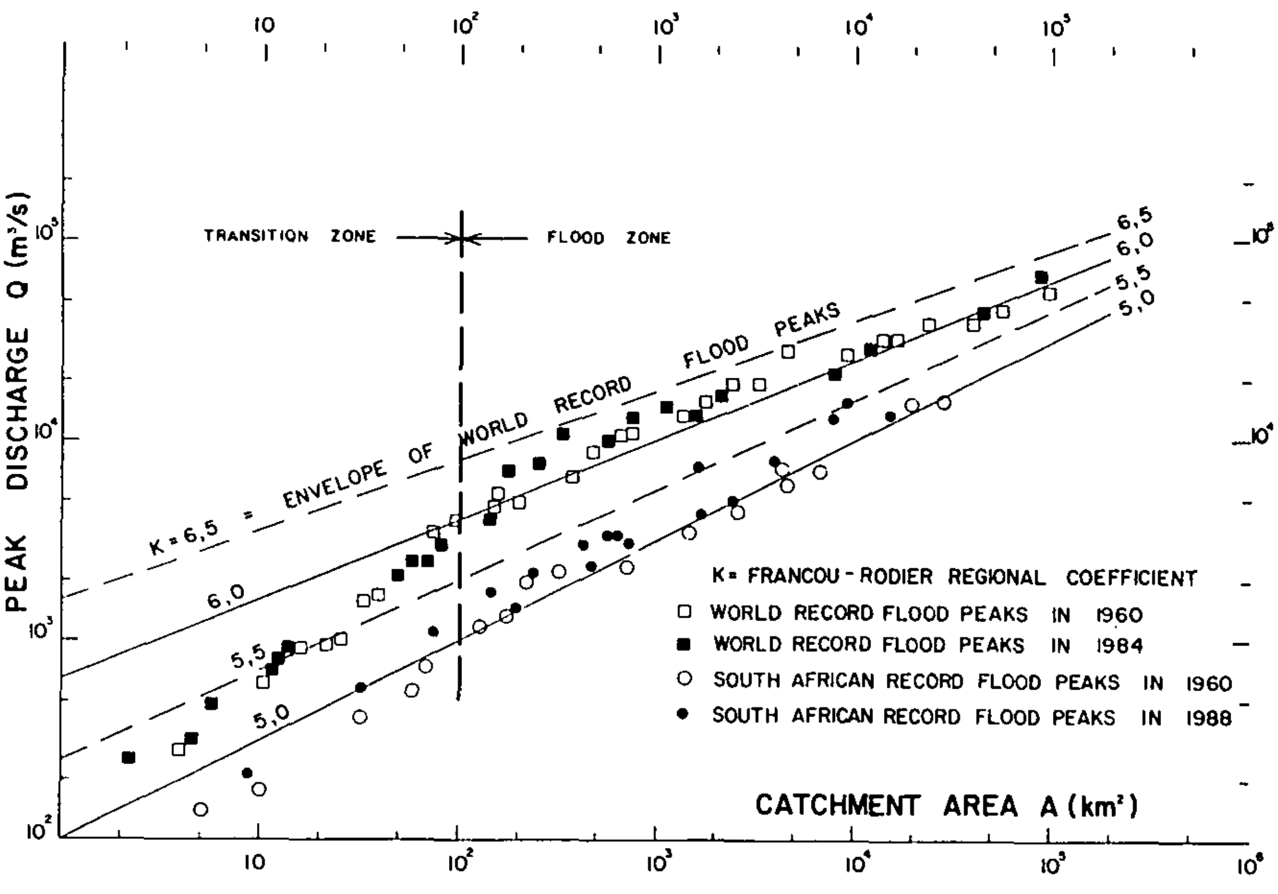


Figure 10.1 *Francou-Rodier envelope plot of maximum observed floods (from Kovacs, 1988b)*

catchment wetness: the outcome is to underestimate seriously the coefficient of variation of flood magnitude. He cites increasing evidence that, for large catchments with response times exceeding 24 hours, it is typically multiple storm events which are critical: prior rainfall — spread over days, weeks or even months — amplifies peaks produced by subsequent storms and leads to abnormally high peaks relative to the magnitude of the storm rainfall.

Alexander's main reservation about the statistical approach appears to concern the treatment of outliers, although his argument that "regionalisation does not resolve the problem where some stations in the region have high outliers while most stations have not" is hard to follow. However, a particularly perceptive point (personal communication from Alexander, 1988) was to ask if the motivation for fitting flood frequency curves by the method of Probability Weighted Moments (Hosking *et al.*, 1984) was the view that earlier methods were overestimating long return period flood peaks due to the larger weight given to high outliers. His fear that this might be the motivation serves to highlight the gulfs between those who see outliers as exceptions to be largely ignored (e.g. Cluckie & Pessoa, 1990), those who give weight to high-class statistical arguments of robustness and down-weight the largest observations almost incidentally (Hosking & Wallis, 1991), and those (like Alexander) who are concerned that the high outliers should in some sense be accommodated.

10.6 Australia

Hazard categories

Implementation of the Dams Safety Act (1978) is monitored by state Dams Safety Committees which report annually. Dams are always prescribed under the Act if they exceed 15 m in height, if they exceed 10 m in height and impound more than 250,000 m^3, or if they exceed 5 m in height and impound more than 500,000 m^3. Lesser dams are prescribed only if they present a high or significant hazard (see below).

The Australian National Committee on Large Dams (ANCOLD) produced interim guidelines on design floods in 1984 which were subsequently confirmed (ANCOLD, 1986). The guidelines recognise three categories of dam according to whether the incremental flood hazard (IFH) posed is high, significant or low. The term 'incremental' implies that one assesses the incremental loss of life, property and services which is attributable to the dam, i.e. over and above that which would occur if the dam did not fail.

Laurenson (1987) provides a useful guide to flood frequency terminology. Australian practice favours quoting the rarity of a design flood in terms of its annual exceedance probability rather than return period. This phraseology is not without merit (see Reed, 1991, for a discussion): however, Table 10.2 summarises the flood standards in the return period form familiar to UK reservoir engineers.

Table 10.2 *Reservoir design floods: Australian practice*

IFH category	Hazard	Return period range
High	Loss of life Extreme damage	10 000 years to PMF
Significant	Loss of life unlikely Significant damage	1000 to 10 000 years
Low	No loss of life Minor damage	100 to 1000 years

A range of return periods is given for each IFH "to allow the owner to exercise judgement in assessing the circumstances of each case" (Cantwell & Murley, 1986).

Reconciling T-year flood and PMF estimates

ANCOLD advises that PMF estimates should be based on PMP estimates prepared using Bureau of Meteorology methods, favouring the generalised methods rather than those based on limited transposition or maximisation *in situ*. The guidelines refer to the new edition of Australian Rainfall Runoff (Institution of Engineers, 1987), to which Pilgrim (1986) provides a summary. The basis of the Australian Rainfall Runoff (ARR) recommendations for estimating rare floods is presented by Rowbottom *et al.* (1986). Of particular interest is the procedure developed for reconciling T-year flood estimates with the Probable Maximum Flood.

Rowbottom *et al.* (1986) recognise that estimates of PMF can derive from various types of PMP estimates, some of which are inherently more conservative than others. A return period is assigned to the PMF, not expressly to label the degree of conservatism but for the purpose of unifying T-year flood and PMF estimates. The assignment takes into account both the relationship of the 50 and 100-year flood estimates to that of the PMF, and the degree of maximisation and transposition implicit in the PMP estimate. There is clearly something to be said for a procedure which reconciles estimates of rare floods derived by statistical analysis (whether of peak flows or through a rainfall-runoff technique) with those obtained by maximisation methods.

Estimates of T-year floods are generally calculated as a weighted average of flood frequency curves derived by statistical and rainfall-runoff methods: greater weight is given to the statistical analysis of peak flows at low return periods and to the rainfall-runoff analysis at high return periods. Estimates of PMF are derived from PMP using either a unit hydrograph or a runoff routing method.

A probabilistic version of the Rational Method

Another novelty of the revised ARR is a rebirth of the Rational Method in which the runoff coefficient, c, is defined as the ratio of the runoff and rainfall frequency curves. The recommendation to obtain values of c at ungauged sites from contoured maps (of c) would appear to be too empirical and coarse. However, the procedure is not intended for use in reservoir flood safety appraisals.

10.7 United States of America

There are numerous agencies concerned with reservoir safety in the USA and it is practical only to touch on a few aspects of reservoir floods research there.

How rare is the PMF?

An interagency committee was set up in 1984 to review the feasibility of assigning a probability to the probable maximum flood.

At first sight this is a curious objective, the more so when viewed from the definition of PMF prevalent in the UK: "The smallest flood that couldn't happen". Present UK practice does not explicitly recognise the risk of occurrence of a flood approaching or exceeding the PMF. However, in other climates and institutional settings it is recognised that the degree of conservatism in PMF estimates is not uniform (Wang & Revell, 1983). In part this is due to the differing assumptions made but in part it may reflect that a standard method may not be equally relevant in all cases. For example, a PMF estimate based on all-year PMP and a snowmelt allowance will clearly be more conservative than one based on summer PMP alone. Also, the neglect of a snowmelt allowance will be less conservative in a region where snow accumulations are commonplace than in one where they are rare.

The interagency working group concluded: "It is not within the state of the art to calculate the probability of PMF-scale floods within definable confidence or error bounds". This conclusion is neither surprising nor helpful. Nevertheless, their report (Interagency Advisory Committee on Water Data, 1986) provides a useful summary of US approaches to PMF and extreme flood estimation.

The report recognises five approaches:

- extrapolation of flood frequency curve,
- combination of frequency distributions of causal factors (e.g. antecedent reservoir level and storm rainfall),
- regional approach to extrapolation (e.g. the station-year method),
- palæoflood analysis (e.g. inferring historical flood levels by the position and dating of sediments), and
- Bayesian analysis (combining different sources of flood data, e.g. local, regional and historical).

It is immediately obvious that flood frequency methods play a role in maximum flood estimation in the USA which is effectively barred in the UK by the recommendation that reservoir design floods should always be based on a rainfall-runoff approach (see Section 2.6).

Conspicuously absent from the list are envelope methods, although an appendix makes comparisons between some observed extreme floods and calculated PMFs for 17 'hydrologic' regions. Although envelope methods are essentially non-statistical — and certainly unscientific in their neglect of catchment factors other than area — the frequency with which records are broken can itself be subjected to statistical analysis and might conceivably form the basis of assigning a return period to the PMF. In discussion to Wang & Revell (1983), Cecilio confirms that it is common practice to compare derived PMF values with a flood envelope method such as Craeger's curves to provide a subjective assessment of the conservatism of the PMF estimate.

As in UK practice, the use of a rainfall-runoff method based on the unit hydrograph is favoured: Wang & Jawed (1986) provide examples. While use of a catchment model to convert PMP into PMF is a valid step towards determinism, the selection of associated initial conditions calls for judgement, which at best will be probabilistic and at worst may be entirely subjective. The assumption of an antecedent storm is usual and Wang & Revell (1983) indicate that a typical depth for this is 40 per cent of the PMP.

The assessment of PMP is a science or art in itself and a review of methods is beyond the scope of the present study. However, it is noted that US practice is relatively sophisticated, with specific maximisations for subject sites.

Summary
It is recognised that US approaches to estimating spillway design floods are much more varied than those in UK practice. In part this arises from the more diverse climatic and physiographic conditions; however, in part it may reflect the weaker institutionalism of reservoir flood estimation in the USA. The narrower range of techniques used in the UK probably leads to a greater consistency in spillway design. However, the greater appetite for explicit risk assessment is perhaps a compensating strength in US practice. This topic is touched on briefly below.

10.8 Research on reservoir risks

A number of recent studies relate to risk-based safety evaluations of dams. Some deal with several aspects of dam safety (Safety and Reliability Directorate, 1985; Bossman-Aggrey *et al.*, 1987) while others are more specific (e.g. National Research Council, 1985; Wellington, 1988). Oosthuizen & Elges (1988) recognise the not unfamiliar situation that not all dams comply with a nation's current safety criteria. Their work attempts to promote the use of risk-based evaluation methods — including dam-break flood modelling — so that those dams with the greatest hazard potential can be upgraded first.

Charles *et al.* (1991) present an engineering guide to seismic risk to dams in the UK. A Royal Society study group report provides an excellent general reference to risk estimation in engineering, particularly with regard to the psychology of risk perception and acceptability (Royal Society, 1983).

An earlier Institute of Hydrology study for the Department of the Environment provides a method for evaluating the risk of an extreme rainfall occurring at one of a network of critical sites (Dales & Reed, 1989). While the immediate contribution to reservoir flood risk assessment was fairly limited, redeployment of the spatial dependence model has led to a new method of pooling rainfall data to obtain frequency estimates of very rare events (Reed & Stewart, 1989a). Wider application of this focused rainfall growth estimation (FORGE) technique should lead to a revision to the FSR procedure for calculating 1000 and 10,000-year rainfall depths, relevant to reservoir flood design. Initial work has developed such estimates for south west England.

11 Summary

The assessment of flood risk is a vital element in the safe design, maintenance and operation of impounding reservoirs. Since the introduction of reservoir safety legislation in 1930, further strengthened by the 1975 Reservoirs Act, the record of reservoir flood safety in the UK has been excellent by world standards. Nevertheless, the need remains to maintain adequate provision to discharge floods safely at some 2400 large impounding reservoirs, many of them more than 100 years old, often sited above the communities which they serve.

This report has re-examined reservoir flood estimation. Although reference is made to developments elsewhere, the primary concern is with UK methods and experience. The principal findings are as follows:

1 The currently recommended procedures are defined collectively by several documents. While the methods are unchanged in principle from those set down in the *Flood Studies Report* (NERC, 1975) and the guide to *Floods and Reservoir Safety* (ICE, 1978), there have been important changes in detail and some useful supplementary guidance. The potential for users to overlook developments has been partly remedied in the second edition of the *Floods and Reservoir Safety* guide (ICE, 1989) by a revised preface. Further coherence has been provided by wider penetration of the *Micro-FSR* computer package, which is fully compatible with *Flood Studies Supplementary Reports*.

2 The report examines those aspects of flood estimation methods and research which are most relevant to reservoir flood safety appraisals. Although much of the discussion is intricate, the report comments in simple terms where possible, for example when spelling out anomalies that can arise in practical application of the procedures.

3 Reference to a sample of 187 reservoired catchments in the UK confirms that they are typically small and steep, in marked contrast to the 209 gauged catchments on which the FSR rainfall-runoff method is based.

4 Extensive comparisons of design floods are made for a subsample of 15 reservoired catchments. Comparisons are presented of potential alternative design floods: for example, between 'summer' and 'winter' values of the Probable Maximum Flood (PMF), and between 0.5 PMF and 10,000-year flood estimates.

5 Whereas most of the 15 catchments are judged to be 'summer-critical' in the pre-reservoir condition, several are seen to be 'winter-critical' in the post-reservoir state. This effect arises from the lengthening of the design storm durations to which the systems become sensitive through reservoir routing action. The effect will be more pronounced for middle and lower reservoirs in 'cascades'.

6 The sensitivity of reservoir flood estimates to the precise storm duration assumed is confirmed as slight. The procedure by which the reservoir 'lag' influences the choice of storm duration is also shown to be satisfactory.

7 Seasonal estimates of rainfall frequency have been re-examined. Departures from those factors given in the FSR are found to be generally slight. However, a new table is provided for estimating seasonal probable maximum rainfall as a proportion of all-year probable maximum rainfall for 1, 2, 4 and 8-day durations.

8 Recent research on spatial dependence in rainfall extremes has exposed some regional biases in the FSR rainfall frequency generalisation. Underestimation of rainfall frequency in much of the west country has been confirmed and the analysis suggests that there may be some overestimation in parts of north west England. The National Rivers Authority plans to fund further research to establish a less biased rainfall frequency generalisation for England and Wales.

9 The significance of heavy rainfall in historical UK dam safety incidents is briefly reviewed. While it is confirmed that few incidents were directly attributable to extreme rainfall, 14 out of 31 were associated with prior heavy rainfall. It is conjectured that some failures attributable to latent geotechnical defects are triggered by heavy rainfall, on the principle of 'the straw that broke the camel's back'. It is recommended that meteorological conditions be included in any central archive of UK reservoir safety incidents.

10 Allowances for snowmelt in reservoir flood estimation are critically reviewed. The

evidence that melt rates as high as 5 mm h^{-1} have been observed in north east England (Archer, 1981) is less clear than one would wish: re-analysis suggests that the role of rainfall has been underplayed. However, it is concluded that such high rates of sustained snowmelt runoff are entirely feasible in UK conditions: the doubt concerns only the rarity of such a melt.

11 Given that the snowmelt allowance is to be used in combination with an estimate of probable maximum rainfall, it is not clear that a very rare value should necessarily be stipulated. However, it is concluded that the present procedure is unsatisfactory because the prescribed rate of 1.75 mm h^{-1} is invariant with location. This clearly provides a less conservative design in those snow-prone regions where opportunities for a large melt arise more frequently. While a crude regionalisation could be attempted — for example, by introducing a snowmelt allowance in proportion to the mean number of days on which lying snow is observed — it is recommended that engineering judgement be applied pending the outcome of the 'Joint Probability Studies for Reservoir Flood Safety' project. This is investigating the statistical basis of the combination of inputs recommended in Table 1 of the Floods and Reservoir Safety guide (ICE, 1978). For those who wish to adopt higher snowmelt allowances, irrespective of catchment location, Archer (1984, pp 13-15) makes specific suggestions about how this should be done.

12 Further understanding of rainfall-runoff processes at the catchment scale comes relatively slowly. However, important advances, such as the availability of a more detailed hydrology of soil types (HOST) classification, are reviewed. While Digital Terrain Models and Geographical Information Systems promise much greater automation of flood estimation procedures in years to come, it is recognised that methodologies may also have to develop if professional standards are to be maintained. Greater automation could otherwise lead to less thoughtful flood estimates.

13 Flood hydrology is only one aspect of reservoir safety appraisal and it is understandable that the dam engineering profession should be keen to apply standard methods where possible. However, it is important that local information (in particular on soils, river flows and historical floods) is incorporated into flood estimates where appropriate. Responsibility for ensuring this rests primarily with the 'panel' engineers, appointed under the 1975 Reservoirs Act to promote reservoir safety.

14 Reservoir routing is an integral part of reservoir flood safety appraisals. This report presents new algorithms for 'level-pool' flood routing, along with detailed examples. Approximate formulae are considered for the peak-to-peak lag time and attenuation ratio associated with reservoir routing.

15 A selective review of procedures and developments elsewhere in the world highlight some of the fundamental choices in reservoir flood estimation that can be missed by the strict devotee of the methods given in the *Flood Studies Report* (NERC, 1975) and the *Floods and Reservoir Safety* guide (ICE, 1978). The procedures followed in some countries appear to be naïve, and in others the methods seem a poor imitation of those used in the UK, but there are a number of interesting exceptions.

16 It is of particular interest that in Australia (and to some extent in the United States) an attempt has been made to reconcile T-year flood and PMF estimates. Panel engineers appear evenly divided in their preference for one approach over the other (Clarke & Phillips, 1984). Quantitative comparisons presented herein suggest that there is no general mismatch between the two sets of design standards in use in the UK. However, this finding should not disguise the genuine discomfort that some feel in the concept of a maximum event, and that others feel in 'blind' statistical extrapolation. Advances being made in methods for pooling data to achieve more systematic estimates of very rare events suggest that the statistical methods will persist, if not prevail.

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It is gratifying to report that the earlier "Reservoir flood and storm hazard assessment" project (Contract PECD7/7/135) has borne further fruit, leading to the FORGE method of estimating very low frequency rainfall events. This success is in no small measure due to the recognition by John Phillips that an element of 'seedcorn' research could be accommodated within DOE's Reservoir Safety Research Commission.

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Appendix A Terms of reference under DoE contract PECD 7/7/181:

"To improve methods of flood estimation for reservoired catchments"

Objective

The objective is to improve methods of flood estimation on reservoired catchments with regard to the following:

- rapidly responding catchments
- reservoir lag and design storm duration
- distinction between summer and winter rainfall
- effect of snow melt.

Programme of work to be carried out by the contractor

1 Initially the contractor will obtain information from users of the ICE *Guide to Floods and Reservoir Safety* to highlight areas of uncertainty in applying the *Flood Studies Report* methods to reservoir flood estimation. This information will enable a comparison to be made between current design standards (based upon the probable maximum flood) and earlier methods.

1a The contractor will investigate the correspondence between T-year and maximum flood estimates for selected reservoired catchments and selected gauged catchments, incorporating the findings in the main final report. The *additional* item relates directly to the flood standards table (Table 1) of the ICE Guide to Floods and Reservoir Safety.

2 The contractor will develop a methodology to deal with the problem of synthesising unit hydrographs for rapidly responding catchments, review snow melt allowances, and examine the interaction between 'reservoir lag' and design storm duration in the *Flood Studies Report* procedure for estimating the maximum probable flood.

3 The information obtained from paragraph 1 above may indicate further areas where research is required to improve the existing methods such as suitable factors to allow for seasonal rainfall (summer and winter).

4 The contractor will produce a streamlined procedure for reservoir flood estimation (incorporating both flood synthesis and reservoir routing) in a form suitable for microcomputer calculation.

5 The contractor will provide the Department of the Environment with progress reports every six months for the duration of the contract, and a final report at the end of the contract. In addition a separate report will be provided for item 4 above.

Appendix B Check-list of UK flood estimation procedures

B1 Rainfall-runoff method

The method presently recommended is that defined in FSSR 16. In essence, the rainfall-runoff method includes an explicit allowance for urbanisation but special changes in the design package are recommended for heavily urbanised catchments (URBAN > 0.25). These are indicated in FSSR 16 but some cross-reference to FSSR 5 is still required.

For the low frequency design events typical of reservoir flood safety applications, the chief difference is the use of a rather sharper (i.e. more peaked) design storm profile. The effect of this on spillway sizing is notable in cases where the flood attenuation provided by the reservoir is small.

Advice for the incorporation of local data is given in FSSR 13 but, for the revised rainfall-runoff method, this must be read in conjunction with FSSR 16. Some further suggestions are given by Reed (1987) which relate primarily to the exploitation of non-standard information. Section 2.4 of the present report considers some points that relate specifically to use of local data in reservoir safety applications.

B2 Statistical method

The FSR and the ICE guide to Floods and Reservoir Safety imply that this method should not be used for reservoir safety application (see Section 2.6). The reasons are not stated prominently. It is convenient to argue that a design hydrograph is always required and that this calls for the use of a rainfall-runoff method. However, there are several ways in which flood estimates from different sources can be reconciled. For example, a rainfall-runoff model parameter such as the standard percentage runoff (SPR) might be adjusted so that the flood frequency relationship tallied with a statistical analysis of peak flows. Another approach is to take a peak flow estimate from a statistical analysis (of site or local flood data) and to flesh this out to a design hydrograph by exploiting the standard hydrograph shapes given in FSSR 9.

However, there is another reason why present guidance discourages application of the statistical method to reservoir flood problems. If estimates are tailored closely to local data, there is the danger that extrapolation to the high return periods relevant to reservoir flood design may lead to gross under- or over-design. This fear also attends the use of local data in the rainfall-runoff method (see Section 2.4). However, because of the greater regional homogeneity in extreme rainfalls, and the longer record lengths available for analysis, use of the rainfall-runoff method is preferred for reservoir safety applications. There are also conceptual reasons for this preference: the rainfall-runoff method provides a model of catchment flood formation rather than just a number. The statistical procedures are nevertheless summarised for completeness.

STEP 1: Estimation of mean annual flood from catchment characteristics

The 6-variable equation recommended in FSR I: 4.3.10 remains the basic method in most regions. Alternative equations derived for small catchments are given in FSSR 6; the 3-variable equation using AREA, SAAR and SOIL is attractively simple but recommended only for use where it proves impossible to estimate the standard stream network characteristics (notably STMFRQ).

The 6-variable equation does not include an explicit allowance for urbanisation. A rule is not given for the circumstances in which the FSSR 5 adjustments for urbanisation should be applied in the statistical method, though an URBAN fraction of 0.10 is currently taught. Because an element of the rainfall-runoff method is exploited in the urban adjustment to the statistical method, cross-reference to FSSR 16 is also required.

A separate equation is recommended for use in Hydrometric Areas 36 to 39 (the "Thames, Lee and Essex" equation) which explicitly represents the degree of urbanisation.

STEP 2: Regional growth curve

The regional growth curves presently recommended are those given in FSSR 14. They differ from the FSR I.2.14 growth curves at return periods greater than 100 years. This rescinds the original recommendation to jump from a regional growth curve to a national growth curve at return periods greater than 500 years.

As a consequence of this change, several of the revised growth curves have a marked kink at T=100 years.

For significantly urbanised catchments (say URBAN > 0.10) the growth curve modifications of FSSR 5 are applied. Advice for the incorporation of local data is summarised in FSSR 13.

B3 Regional variations

Where refinements have been developed only for specific regions, these have neither been published in the FSSR series nor implemented in Micro-FSR. This Appendix notes those regional variations known to the authors.

Rainfall-runoff method

A study carried out principally for South West Water led to a revised procedure for estimating T-year, D-hour rainfall depths in South West England (Stewart, 1989b; Reed & Stewart, 1989b).

This research was prompted by the recognition by Dales & Reed (1989) that the FSR rainfall frequency estimation methods are overgeneralised and fail to represent regional variation in rainfall growth rates adequately. Bootman & Willis (1981) had already demonstrated that the FSR method underestimated rainfall growth rates in parts of Somerset. However Dales & Reed's suggestion that the FSR method may over-estimate rainfall depths in north west England was new and has yet to be fully investigated.

Statistical method

STEP 1: Estimation of mean annual flood from catchment characteristics

As part of a reworking of the statistical method in the South West Water region (see below), Whiter (1983) derived a modified coefficient for the 6-variable equation in conjunction with the Institute of Hydrology. This is 0.0284 rather than the original 0.0315.

Catchments from Northern Ireland were not well represented in the original UK Flood Studies project, mainly because of the paucity of long-term flow records. Subsequent analysis by Hanna & Wilcock (1984) permitted derivation of a special equation for estimation of mean annual flood in the province. This takes the form of a modification to the multiplier in a 4-variable equation given in Table I. 4.13b of the FSR: the revised equation is

$$QBAR = 2.0999\times10^{-4}\ AREA^{0.99}\ S1085^{0.37}\ SAAR^{1.11}\ SOIL^{0.96} \qquad \text{B3.1}$$

STEP 2: Regional growth curve

The reworking of the FSR statistical method in the South West Water region undertaken by Whiter (1983) also led to a revised regional growth curve. He analysed 417 station-years of data from 25 stations (i.e. an average record length of 16.7 years). The resultant regional growth curve is defined by a Generalised Extreme Value (GEV) distribution with the parameters u=0.83, a=0.25 and k=-0.24, which is rather steeper than the Region 8 growth curve (u=0.78, a=0.28 and k=-0.10) given in FSR I.2.6.2. (Note that the South West Water region is less extensive than FSR Region 8.) For return periods higher than 100 years, Whiter applies the 'subnational' adjustment procedure recommended in FSSR 14.

A growth curve in use in parts of the Thames basin is that derived by Capel-Davies (Thames Water, 1982) in a design study for flood alleviation works on the Wey. It is sometimes referred to as the Thames regional growth curve, although it is based only on flood data from the Wey, Mole, Rother and Arun basins (the last two being in the neighbouring Southern region). Defined in graphical form, an approximating GEV distribution does not seem to have been derived.

Appendix C Reservoir routing — additional information

C1 Allowance for rain falling on lake area

The formulation of the routing problem is discussed in Section 9.3 of the main report. The solution procedure (Section 9.4) represents the variation in lake area with water level through Equation 9.4 for the purpose of modelling the reservoir routing effect. However, it assumes a fixed area for the purpose of modelling rain falling directly on the lake area: there are two reasons for this.

Firstly, it is sometimes convenient to ignore the direct rainfall effect. For example, one may wish to check a reservoir routing calculation that is known not to allow explicitly for this effect. Secondly, where the effect *is* represented, it is highly inconvenient to have to calculate the inflow hydrograph to the reservoir for a variable catchment area (the area decreasing slightly as the lake area expands during passage of the flood). Using a fixed area is therefore preferable.

Should the rate of change of lake area with water level be significant in terms of the direct rainfall effect, it would be advisable to note the average lake area during passage of the flood and to repeat the calculations using this area as the fixed area for direct rainfall calculations. The Appendix C2 software permits the area to be used for direct rainfall to be specified independently from that used in the reservoir routing. (This dual specification of lake area is not implemented in version 2 of Micro-FSR, but an experienced user can circumvent this limitation.)

C2 Numerical solution scheme — standard case

The primary notation is introduced in Chapter 9 of the main report. Let h_1 and h_2 denote the water levels at the start and end of the modelling interval, Δt. Similarly, let p_1, p_2 and q_1, q_2 denote the inflow and outflow rates at these times. In addition, let a_f denote the fixed area (m^2) for direct rainfall calculations.

Then a finite difference representation of the routing equation (Equation 9.7) is given in Equation C2.1.

Denoting $r\Delta t$ by R and $0.5(p_1+p_2)\Delta t$ by P, this yields Equation C2.2 on re-arrangement. This is solved for h_2 by Newton-Raphson iteration. A suitable initial approximation for h_2 is $h_2=h_1$.

C3 Numerical solution scheme — transition case

A transition arises when the water level, h_2, at the end of the modelling interval lies outside the range of the rating relationship presently in force. This is checked by reference to a table of all the levels at which the q-h relationship changes. The finite difference representation (Equation C2.1) of the routing equation is then rewritten to determine the time, T, at which the transition water level, h_T, is reached within the modelling interval.

The relevant equation is C3.1, where $p_T = p_1 + (p_2 - p_1)T/\Delta t$. In general this yields a quadratic in T (Equation C3.2).

$$\{a_0 + a_1(h_1 + h_2)/2\}\,\frac{h_2 - h_1}{\Delta t} = \frac{p_1 + p_2}{2} + a_f r - \frac{\Sigma\, bc(h_1 - d)^e + \Sigma\, bc(h_2 - d)^e}{2} \qquad \text{C2.1}$$

$$(h_2 - h_1)\,\{2a_0 + a_1(h_1 + h_2)\} = 2\,(P + a_f R) - \Delta t\,\{\Sigma\, bc(h_1 - d)^e + \Sigma\, bc(h_2 - d)^e\} \qquad \text{C2.2}$$

$$\frac{h_T - h_1}{T} = r + \frac{(p_1 + p_T)/2 - \{\Sigma\, bc(h_1 - d)^e + \Sigma\, bc(h_T - d)^e\}/2}{\{a_0 + a_1(h_1 + h_T)/2\}} \qquad \text{C3.1}$$

$$\frac{(p_2 - p_1)}{\Delta t}T^2 + [2(p_1 + r.a_f) - \{\Sigma\, bc(h_1 - d)^e + \Sigma\, bc(h_T - d)^e\}]T + (h_1 - h_T)\{2a_0 + a_1(h_1 + h_T)\} = 0 \qquad \text{C3.2}$$

$$T = (h_T - h_1)\{2a_0 + a_1(h_1 + h_T)\}/[2(p_1 + r.a_f) - \{\Sigma\, bc(h_1 - d)^e + \Sigma\, bc(h_T - d)^e\}] \qquad \text{C3.3}$$

The solution which lies between 0 and Δt is selected. In the special case when $p_2=p_1$, T is obtained from Equation C3.3.

The standard solution scheme (Appendix C2) is then restarted from part-way through the data interval, using the q-h relationship which applies above (or below) the transition water level, h_T.

C4 Examples

C4.1 Shaw's example

The standard text *Hydrology in Practice* by E.M. Shaw (1988) gives a reservoir flood routing example attributed to Scott-Moncrieff (unpublished Imperial College lecture notes, 1977). It is a relatively simple example which can be represented precisely in the formulation used here. The choice of an inflow hydrograph of triangular shape is, though, somewhat unrealistic.

For the inflow and outflow hydrographs in turn, a peak value and peak time were estimated from the ordinate data by quadratic interpolation; the relevant code is included in Appendix C5. These values form the basis of the peak-to-peak reservoir lag times and attenuation ratios given in Table C2.

C4.2 Wilson's example

The standard text *Engineering Hydrology* by E.M. Wilson (1990) gives a rather complex example. Interestingly, it is flawed: the storage-head relationship used in the routing is inconsistent with the area-head relationship from which it should derive.

In general it seems wise to work only in terms of the area-head relationship, as in the program ROUTER and its implementation within version 2 of Micro-FSR. This is because a mistaken storage-head relationship is difficult to spot whereas a lake area is relatively easy to verify.

Table C1 *Comparison of results: Shaw's example*

Time	Inflow	Outflow (Shaw)	Outflow (ROUTER)	Water level (ROUTER)
h	*$m^3 s^{-1}$*	*$m^3 s^{-1}$*	*$m^3 s^{-1}$*	*m*
0	0	0	0.00	0.00
2	60	1	0.53	0.03
4	120	6	4.10	0.11
6	180	14	13.14	0.24
8	240	31	29.17	0.41
10	300	55	52.69	0.61
12	360	85	83.46	0.83
14	330	114	115.11	1.03
16	300	138	140.61	1.18
18	270	158	159.62	1.28
20	240	171	172.42	1.35
22	210	178	179.55	1.39
24	180	180	181.65	1.40
26	150	178	179.38	1.39
28	120	173	173.36	1.35
30	90	163	164.17	1.31
32	60	152	152.31	1.24
34	30	139	138.27	1.16
36	0	123	122.47	1.07

Shaw's example:

A-h relationship:

$A_0 = 7.5\ km^2$ at $h = 0.0$ m
$A_1 = 1.5\ km^2\ m^{-1}$
$A_\gamma = 0.0$
Notation A_i rather than a_i refers to area in km^2 not m^2.
No explicit allowance is made for direct rainfall in this example.

q-h relationship:

Device 1, Equation 1 (free spillway): $bc = 110\ m^{1.5}\ s^{-1}$
$h_{min} = 0.0$ m; $d = 0.0$ m
$h_{max} = 9999$ m (i.e. unlimited); $e = 1.5$

Initial condition: Reservoir just full, i.e. $h = 0.0$ m.

Results: Table C1 defines the inflow hydrograph and compares the outflow simulation provided by the ROUTER program (Appendix C5) with that given by Shaw. There are some differences but these are not large.

Table C2 *Attenuation of peak inflow, Shaw's example*

	Peak-to-peak lag time RLAG (h)	Attenuation ratio α_1
Shaw	11.67	0.498
ROUTER	11.63	0.503

Wilson's example:

A-h relationship:

$A_0 = 4.504\ km^2$ at $h = 63.0$ m
$A_1 = 0.118\ km^2\ m^{-1}$
$A_\gamma = 0.0$ (no explicit allowance for direct rainfall in this example)

q-h relationship:

Device 1, Equation 1 (twin circular ports, part-full)
$h_{min} = 52.65$ m; $h_{max} = 55.35$ m
$bc = 17.461\ m^2\ s^{-1}$ (chosen to match Equation 2 at $h = 55.35$ m)
$d = 52.65$ m; $e = 1.0$

Device 1, Equation 2 (twin circular ports, drowned)
$h_{min} = 55.35$ m; $h_{max} = 9999$ m (i.e. unlimited)
$b = 50.72\ m^{2.5}\ s^{-1}$; $c = 0.8$
$d = 54.0$ m; $e = 0.5$

Device 2, Equation 1 (free spillway)
$h_{min} = 66.0$ m; $h_{max} = 9999$ m (i.e. unlimited)
$b = 72.5$ m; $c = 2.2\ m^{0.5}\ s^{-1}$
$d = 66.0$ m; $e = 1.5$

Initial condition: Reservoir water level is 63.5 m, i.e. 9.5 m above the centre line of the twin ports but 2.5 m below the main spillway.

Results: Table C3 defines the inflow hydrograph and compares the results of the ROUTER program (Appendix C5) with the results given by Wilson in later editions of his text (the 1st edition contained errors). Also shown in Table C3 are results using a reservoir routing program believed to correspond to that listed in I.7.4.2 of the FSR.

Table C3 *Comparison of results: Wilson's example*

Time	Inflow	Outflow (Wilson)	Outflow (FSR)	Outflow (ROUTER)	Water level (ROUTER)
h	*$m^3 s^{-1}$*	*$m^3 s^{-1}$*	*$m^3 s^{-1}$*	*$m^3 s^{-1}$*	*m*
0	50	125	125.26	125.07	63.50
6	75	122	123.35	123.13	63.21
12	180	122	123.48	123.26	63.23
18	350	127	127.74	127.59	63.89
24	450	136	135.56	135.35	65.13
30	520	200	209.39	217.82	66.60
36	505	425	405.03	422.36	67.43
42	445	460	456.89	463.47	67.57
48	360	416	416.42	415.95	67.41
54	290	347	351.35	347.94	67.17
60	250	288	296.74	292.96	66.95
66	210	242	254.89	251.33	66.76
72	175	208	218.72	215.27	66.58
78	140	190	186.78	183.39	66.40
84	110	165	157.00	156.33	66.20
90	85	144	140.60	140.45	65.98
96	65	140	138.82	138.75	65.69
102	55	138	136.66	136.66	65.34
108	50	134	134.31	134.37	64.97
114	45	132	131.84	131.94	64.57
120	40	129	129.25	129.37	64.16
126	38	127	126.59	126.69	63.75

Table C4 *Attenuation of peak inflow: Wilson's example*

	Peak-to-peak lag time RLAG (h)	Attenuation ratio α_1
Wilson	9.72	0.877
FSR	10.43	0.872
ROUTER	9.84	0.884

However, so that comparisons can be made with Wilson and the Flood Studies Report (which also uses this example), an area-head relationship has been derived which corresponds to the storage-head relationship actually used by Wilson. The A-h relationship used below provides a good fit to Wilson's S-h relationship over the 63 to 68 m water level range relevant to the problem.

Table C5 *Rating table: Wilson's example*

Water level m	Outflow rate $m^3 s^{-1}$	Water level m	Outflow rate $m^3 s^{-1}$	Water level m	Outflow rate $m^3 s^{-1}$
58.0	81.2	66.1	146.2	66.7	238.0
60.0	99.4	66.2	156.0	66.9	281.9
62.0	114.8	66.3	168.5	67.0	305.8
64.0	128.3	66.4	183.2	67.5	442.1
66.0	140.6	66.5	199.9	68.0	603.0

Table C6 *Comparison of outflows: Wilson's example*

Time h	Equation option $m^3 s^{-1}$	Table option $m^3 s^{-1}$	Time h	Equation option $m^3 s^{-1}$	Table option $m^3 s^{-1}$
0	125.07	124.92	66	251.33	251.69
6	123.13	122.95	72	215.27	215.96
12	123.26	123.10	78	183.39	183.35
18	127.59	127.55	84	156.33	156.54
24	135.35	135.24	90	140.45	140.50
30	217.82	220.75	96	138.75	138.73
36	422.36	418.36	102	136.66	136.58
42	463.47	457.33	108	134.37	134.27
48	415.95	413.63	114	131.94	131.85
54	347.94	348.00	120	129.37	129.35
60	292.96	292.43	126	126.69	126.64

The corresponding peak-to-peak lag times and attenuation ratios are given in Table C4. The discrepancies are fairly minor but the FSR outflow hydrograph is notably later peaked. The differences are not explained by the particular representation here of the twin circular ports in the undrowned condition, since the initial water level is well above this range.

C4.3 Example of 'rating table' option

Wilson's example can be used to illustrate the 'rating table' option, where the q-h relationship is specified by data points. Using the program ROUTER, the effect of specifying by rating table can be compared with the normal specification by rating equations. Table C6 confirms that the table option provides a reasonable representation in this instance. The more accurate solution is of course the one based on the equations.

C5 Program listing

A listing of the Fortran program ROUTER follows. A copy of the code can be supplied as an ASCII file on a 3.5" diskette from the first author at the Institute of Hydrology. A listing of a simple program for peak interpolation is also attached.

Program ROUTER differs from the routing software implemented in version 2 of Micro-FSR in three respects. Firstly, Micro-FSR provides user-friendly data entry screens which carry out some of ROUTER's functions and checks prior to execution of the hydrograph routing. Secondly, ROUTER permits the lake area used for direct rainfall calculations to be specified independently from that used in the reservoir routing (see Appendix C1). Finally, in order to provide additional flexibility for balancing pond design, Micro-FSR uses a more general form of area-head relationship.

```
C    ***   DUNCAN'S PREFERRED RESERVOIR ROUTING PROGRAM    ***
C    ***   AMENDED 15 OCT AND 13 DEC 90 TO INCORPORATE REVISONS    ***
C    ***   IDENTIFIED IN MICROFSR IMPLEMENTATION    ***
C    ******UNITS**************************************
C    ALL FLOWS (EG. QIN, QOUT) IN CUMECS
C    ALL LEVELS (EG. WLEVEL, HMIN) IN M
C    DATA INTERVAL IN H (DT) OR S (DELTAT)
C    AREAS IN SQ.KM. (EG. ARAIN) OR SQ.M. (EG. AREA)
C    ************************************************
      DIMENSION QIN(500),QOUT(500),RAIN(500),WLEVEL(500)
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      HTOL=0.0005
      HTOL2=0.0001
      QTOL=0.001
      ILIM=20
C    ***   READ RESERVOIR CHARACTERISTICS    ***
      CALL RDRES
C    ***   READ DATA INTERVAL    ***
      READ(1,*)DT
      DELTAT=DT*3600
      TTOL=0.0001*DELTAT
C    ***   READ INFLOW AND RAINFALL DATA    ***
      READ(1,*)N
      READ(1,*)(RAIN(I),QIN(I),I=1,N)
      WRITE(6,959)
  959 FORMAT('0ENTER RESERVOIR AREA (SQ.KM.) TO BE USED IN',
     1' CALCULATION OF DIRECT RAINFALL (CAN BE ZERO)')
      READ(5,*)ARAIN
      AREA=1000000*ARAIN
      WRITE(6,960)
  960 FORMAT('0ENTER INITIALIZATION OPTION FOR RESERVOIR WATER LEVEL',
     1'/OUTFLOW'/' 0 = OUTFLOW CORRESPONDS TO BASEFLOW'/
     2' 1 = OUTFLOW SPECIFIED BY USER'/
     3' 2 = WATER LEVEL SPECIFIED BY USER'/)
C    ***   INITIALIZE WATER LEVEL AND OUTFLOW    ***
      DEPLET=0.0
      DTOL=ALOW*HTOL2
      READ(5,*)INIOPT
      IF (INIOPT.EQ.0) GOTO 24
      IF (INIOPT.EQ.1) GOTO 22
C    ***   USER SPECIFIES INITIAL WATER LEVEL    ***
      READ(5,*)H0
      IF (H0.GE.HCRIT(1)) GOTO 20
C    ***   RESERVOIR IS INITIALLY DEPLETED    ***
      DEPLET=(HCRIT(1)-H0)*(A0+A1*0.5*(H0+HCRIT(1)))
      Q0=0.0
      GOTO 28
   20 CONTINUE
      Q0=QFROMH(H0)
      GOTO 28
   22 CONTINUE
```

```
C   ***   USER SPECIFIES INITIAL OUTFLOW   ***
      READ(5,*)Q0
      GOTO 26
   24 CONTINUE
C   ***   RESERVOIR ASSUMED TO BE INITIALLY SPILLING   ***
C   ***   AT RATE OF INCOMING BASEFLOW   ***
      Q0=QIN(1)
   26 CONTINUE
C   ***   DETERMINE CORRESPONDING WATER LEVEL   ***
      H0=HFROMQ(Q0)
   28 CONTINUE
      QOUT(1)=Q0
      WLEVEL(1)=H0
C   ***   PREPARE TO START ROUTING   ***
      H1=H0
C   ***   FIND OUTFLOW RANGE   ***
      IR1=IRFRMH(H1)
C   ***   MAIN TIME LOOP   ***
      DO 50 IT=1,N-1
C   ***   CHECK IF WATER LEVEL AT START OF DATA INTERVAL
C         IS AT, OR VERY CLOSE TO, A TRANSITION   ***
      INTRAN=0
      DO 30 IR=1,NR
      IF (ABS(H1-HCRIT(IR)).LT.HTOL2) INTRAN=1
   30 CONTINUE
C   ***   INTRAN=1 INDICATES THAT WATER LEVEL AT START OF DATA
C         INTERVAL IS AT, OR VERY CLOSE TO, A TRANSITION   ***
   33 CONTINUE
      TLAPSE=0.0
   35 CONTINUE
      P1=QIN(IT)+(QIN(IT+1)-QIN(IT))*TLAPSE/DELTAT
      P2=QIN(IT+1)
      R=0.001*RAIN(IT)/DELTAT
      IF (DEPLET.LT.DTOL) GOTO 39
C   ***   RESERVOIR DEPLETED   ***
      WRITE(3,9689)IT,TLAPSE,P1,P2,DEPLET
 9689 FORMAT(' D',I4,4F12.2)
      CALL REPLEN(DEPLET,P1,P2,R,DELTAT,TLAPSE)
      H2=HFROMD(DEPLET)
      IF (DEPLET.GT.DTOL) GOTO 49
C   ***   RESERVOIR REFILLS   ***
      IR1=2
      H1=HCRIT(1)
      GOTO 35
   39 CONTINUE
      CALL ROUTE(P1,P2,R,H1,H2,DELTAT-TLAPSE,IR1)
      WRITE(3,968)IT,TLAPSE,H1,H2,QFROMH(H2),IR1
  968 FORMAT(' R',I4,F12.2,2F12.4,F12.2,I4)
      IR2=IRFRMH(H2)
C   ***   CHECK FOR TRANSITION IN OUTFLOW   ***
      IF (IR1.EQ.IR2) GOTO 49
C   ***   THERE IS A TRANSITION   ***
C   ***   IF WE ARE STILL AT THE START OF THE DATA INTERVAL AND
C         THE WATER LEVEL WAS KNOWN TO BE AT, OR VERY NEAR TO,
C         A TRANSITION, PREPARE TO MAKE THE CHANGE THERE   ***
```

```
      IF (INTRAN.EQ.0.OR.TLAPSE.GT.TTOL) GOTO 42
      IR1=IR2
      INTRAN=0
      GOTO 33
   42 CONTINUE
      IF (IR1.GT.IR2) GOTO 44
C   ***   TRANSITION TO HIGHER OUTFLOW RANGE   ***
      IR2=IR1+1
      HT=HCRIT(IR1)
      GOTO 46
   44 CONTINUE
C   ***   TRANSITION TO LOWER OUTFLOW RANGE   ***
      IR2=IR1-1
      HT=HCRIT(IR2)
   46 CONTINUE
C   ***   ROUTE UP TO TIME OF TRANSITION   ***
      CALL TRANS(P1,P2,R,H1,HT,DELTAT-TLAPSE,T,IR1)
C   ***   UPDATE POSITION IN DATA INTERVAL   ***
      TLAPSE=TLAPSE+T
      WRITE(3,961)HT,T/3600.0,TLAPSE/3600.0,DELTAT/3600.0
  961 FORMAT(' OUTFLOW REACHES TRANSITION WATER LEVEL OF',F8.4,' AFTER',
     1F8.3,' HOURS OR',F8.3,' HOURS INTO',F8.3,' HOUR DATA INTERVAL')
      QT=QFROMH(HT)
      WRITE(3,969)IT,TLAPSE,H1,HT,QT,IR1
  969 FORMAT(' T',I4,F12.2,2F12.4,F12.2,I4)
C   ***   PREPARE TO ROUTE INFLOW FOR REMAINDER OF DATA INTERVAL   ***
      H1=HT
      IR1=IR2
C   ***   RETURN TO REUSE CODE   ***
      GOTO 35
   49 CONTINUE
C   ***   PREPARE FOR NEXT TIME STEP   ***
      QOUT(IT+1)=QFROMH(H2)
      WLEVEL(IT+1)=H2
      WRITE(3,999)IT,H2,QOUT(IT+1)
  999 FORMAT(I12,2F12.4)
      H1=H2
   50 CONTINUE
C   ***   REPORT ROUTING   ***
      WRITE(6,962)(I,(I-1)*DT,RAIN(I),QIN(I),QOUT(I),WLEVEL(I),I=1,N)
  962 FORMAT('1INTERVAL  TIME      RAIN    INFLOW  OUTFLOW   WATER LEVEL'
     1/'              H        MM    MM**3/S  MM**3/S         M'/
     2250(I6,4F9.2,F12.2/),250(I6,4F9.2,F12.2/))
      STOP
      END

      SUBROUTINE RDRES
C   ***   READS AND EVALUATES RESERVOIR CHARACTERISTICS   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20),HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      DIMENSION HSET(40),HMAX(20),B(20),C(20)
C   ***   READ RESERVOIR STORAGE PARAMETERS   ***
      READ(2,*)HSURV,ASURV,AGR
```

```
      A0=1000000*(ASURV-HSURV*AGR)
      A1=1000000*AGR
C   ***   OPTION FOR DEFINITION OF OUTFLOW RATING   ***
      READ(2,*)IROPT
      IF (IROPT.EQ.0) GOTO 291
C   ***   READ DETAILS OF RATING CURVES   ***
      READ(2,*)ND
      READ(2,*)(HMIN(ID),HMAX(ID),B(ID),C(ID),D(ID),E(ID),ID=1,ND)
C   ***   VERIFY THAT RATING LIMITS ARE VALID   ***
      DO 210 ID=1,ND
      IF (ABS(D(ID)-HMIN(ID)).LT.HTOL2) GOTO 210
      WRITE(6,921)ID,D(ID),HMIN(ID)
  921 FORMAT('OWARNING - RATING CURVE',I4,' HAS DATUM OF', F8.4,
     1' BUT LOWER LIMIT OF',F8.4)
      IF (HMIN(ID).GT.D(ID)) GOTO 210
      WRITE(6,9215)
 9215 FORMAT('ODATUM MUST BE LESS THAN LOWER LIMIT')
      STOP
  210 CONTINUE
C   ***   FORM "BC" PARAMETER   ***
      DO 220 ID=1,ND
  220 BC(ID)=B(ID)*C(ID)
C   ***   IDENTIFY THRESHOLD WATER LEVELS   ***
      DO 230 ID=1,ND
      IR=2*ID-1
      HSET(IR)=HMIN(ID)
      HSET(IR+1)=HMAX(ID)
  230 CONTINUE
      NR=IR+1
C   ***   ORDER THRESHOLD LEVELS   ***
      DO 240 I=1,NR-1
      DO 240 J=I+1,NR
      IF (HSET(I).LE.HSET(J)) GOTO 240
      HTEMP=HSET(I)
      HSET(I)=HSET(J)
      HSET(J)=HTEMP
  240 CONTINUE
C   ***   ELIMINATE DUPLICATE DEFAULT MAXIMUM LEVELS   ***
      DO 245 I=2,NR
      IR=I
      IF (HSET(IR).GT.HSET(IR-1)+100.0) GOTO 246
  245 CONTINUE
  246 NR=IR
      WRITE(3,924)NR,(HSET(IR),IR=1,NR)
  924 FORMAT(I4,4(8F10.3/))
C   ***   ELIMINATE OTHER DUPLICATE LEVELS; SET UP "HCRIT"   ***
      IR=1
      HCRIT(IR)=HSET(1)
      DO 250 I=2,NR
      IF (ABS(HCRIT(IR)-HSET(I)).LT.HTOL) GOTO 250
      IR=IR+1
      HCRIT(IR)=HSET(I)
  250 CONTINUE
      NR=IR
      WRITE(3,924)NR,(HCRIT(IR),IR=1,NR)
```

```
C   ***   CHECK THAT AT LEAST TWO FLOW RANGES   ***
      IF (NR.GE.2) GOTO 251
      WRITE(6,922)NR
  922 FORMAT('OERROR - ONLY',I4,' FLOW RANGE')
      STOP
  251 CONTINUE
C   ***   IDENTIFY CORRESPONDING THRESHOLD OUTFLOWS   ***
C   ***   AND SET UP REFERENCE ARRAY "IREF"   ***
      DO 290 IR=1,NR
      H=HCRIT(IR)-HTOL2
      QSUM=0.0
      DO 270 ID=1,ND
      IREF(IR,ID)=0
      IF (H.LT.HMIN(ID).OR.H.GT.HMAX(ID)) GOTO 270
      QSUM=QSUM+BC(ID)*(H-D(ID))**E(ID)
      IREF(IR,ID)=1
  270 CONTINUE
      H=HCRIT(IR)+HTOL2
      QQSUM=0.0
      DO 280 ID=1,ND
  280 IF (H.GE.HMIN(ID).AND.H.LE.HMAX(ID))
     1QQSUM=QQSUM+BC(ID)*(H-D(ID))**E(ID)
C   ***   REPORT ANY DISCONTINUITY   ***
      QQTOL=QTOL*0.5*(QQSUM+QSUM)
      IF (ABS(QQSUM-QSUM).GT.QQTOL) WRITE(6,923)QSUM,QQSUM,HCRIT(IR)
  923 FORMAT('OWARNING - OUTFLOW DISCONTINUITY FROM',F12.4,' TO',F12.4,
     1' AT WATER LEVEL OF',F8.4)
      QCRIT(IR)=0.5*(QSUM+QQSUM)
  290 CONTINUE
      GOTO 298
  291 CONTINUE
C   ***   READ DATA POINTS DEFINING OUTFLOW RATING   ***
      READ(2,*)NR
      IF (NR.LT.2.OR.NR.GT.20) GOTO 299
      READ(2,*)(HCRIT(IR),QCRIT(IR),IR=1,NR)
      IF (QCRIT(1).LT.QTOL) GOTO 293
      WRITE(6,928)
  928 FORMAT('OWARNING - ZERO FLOW LEVEL UNDEFINED')
  293 CONTINUE
      HCRIT(NR+1)=9999
      QCRIT(NR+1)=999999
C   ***   INITIALIZE REFERENCE ARRAY   ***
      ND=NR-1
      DO 294 IR=1,NR
      DO 294 ID=1,ND
  294 IREF(IR,ID)=0
C   ***   EVALUATE RATING CURVE PARAMETERS   ***
      DO 296 IR=2,NR
      ID=IR-1
      HH=HCRIT(IR-1)
      QQ=QCRIT(IR-1)
      IF (ABS(HCRIT(IR)-HH).LT.HTOL2) GOTO 2995
      XX=(QCRIT(IR)-QQ)/(HCRIT(IR)-HH)
      E(ID)=1.0
      D(ID)=HH-QQ/XX
```

```
      HMIN(ID)=HH
      BC(ID)=XX
      IREF(IR,ID)=1
  296 CONTINUE
  298 CONTINUE
C   ***   NOTE LAKE AREA AT LOWEST DISCHARGE LEVEL   ***
      ALOW=1000000*(ASURV+(HCRIT(1)-HSURV)*AGR)
C   ***   REPORT REFERENCE ARRAY   ***
      WRITE(3,925)
  925 FORMAT('OREFERENCE ARRAY')
      DO 2985 ID=1,ND
 2985 WRITE(3,926)(IREF(IR,ID),IR=1,NR)
  926 FORMAT(25I2)
      WRITE(3,927)ND,(HMIN(ID),BC(ID),D(ID),E(ID),ID=1,ND)
  927 FORMAT(I4/20(4F12.4/))
      RETURN
  299 CONTINUE
C   ***   ERROR IN SPECIFYING OUTFLOW RATING   ***
      WRITE(6,929)NR
  929 FORMAT('OFAILURE - NUMBER OF POINTS SPECIFIED IN OUTFLOW RATING',
     1' WAS',I4,' BUT NEEDS TO BE IN RANGE 2 TO 20')
      STOP
 2995 CONTINUE
      WRITE(6,9295)
 9295 FORMAT('OFAILURE - OUTFLOW RATING ILL-DEFINED')
      STOP
      END
C
      SUBROUTINE REPLEN (DEPLET,P1,P2,R,DELTAT,TLAPSE)
C   ***   INFLOW REPLENISHES RESERVOIR, REDUCING DEPLETION   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      DEPINI=DEPLET
      VOL=DELTAT*(0.5*(P1+P2)+R*AREA)
      DEPLET=DEPLET-VOL
      IF (DEPLET.GT.DTOL) RETURN
      TLAPSE=DELTAT*DEPINI/VOL
      DEPLET=0.0
      RETURN
      END
C
      REAL FUNCTION QFROMH (H)
C   ***   CALCULATES Q CORRESPONDING TO GIVEN H   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      IF (H.LT.HCRIT(1)-HTOL2) GOTO 845
      IR=2
  801 CONTINUE
      IF (H.GE.HCRIT(IR-1).AND.H.LT.HCRIT(IR)) GOTO 815
```

```
      IR=IR+1
      IF (IR.LE.NR) GOTO 801
      WRITE(6,981)H
  981 FORMAT('OWARNING - GIVEN WATER LEVEL',F8.4,' IS OUTSIDE EXPECTED',
     1' RANGE')
  815 CONTINUE
      Q=0
      DO 830 ID=1,ND
      IF (IREF(IR,ID).EQ.1.AND.H.GT.D(ID)+HTOL2)
     1Q=Q+BC(ID)*(H-D(ID))**E(ID)
  830 CONTINUE
      QFROMH=Q
      RETURN
  845 CONTINUE
C   ***   RESERVOIR DEPLETED   ***
      QFROMH=0.0
      RETURN
      END
C
      REAL FUNCTION HFROMQ (Q)
C   ***   CALCULATES WATER LEVEL CORRESPONDING TO GIVEN OUTFLOW   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      IF (Q.GT.QCRIT(1)+QTOL) GOTO 715
C   ***   NO OVERFLOW; HENCE NO INFORMATION ON WATER LEVEL   ***
      H=HCRIT(1)
      WRITE(6,971)Q
  971 FORMAT('OWARNING - WATER LEVEL SET TO SILL LEVEL FOR GIVEN',
     1' OUTFLOW OF',F8.4)
      GOTO 765
  715 CONTINUE
      IR=IRFRMQ(Q)
C   ***   SOLVE FOR WATER LEVEL BY NEWTON-RAPHSON ITERATION   ***
      ITER=0
      H=0.5*(HCRIT(IR-1)+HCRIT(IR))
  721 CONTINUE
      FH=-Q
      FDASHH=0.0
      DO 730 ID=1,ND
      IF (IREF(IR,ID).EQ.0.OR.H.LT.D(ID)+HTOL2) GOTO 730
      FH=FH+BC(ID)*(H-D(ID))**E(ID)
      FDASHH=FDASHH+BC(ID)*E(ID)*(H-D(ID))**(E(ID)-1)
  730 CONTINUE
      IF (ABS(FDASHH).LT.HTOL2) GOTO 795
      HNEW=H-FH/FDASHH
      IF (ABS(HNEW-H).LT.HTOL) GOTO 745
      ITER=ITER+1
      H=HNEW
      IF (ITER.LT.ILIM) GOTO 721
C   ***   ITERATION LIMIT EXCEEDED   ***
      WRITE(6,973)H,Q
  973 FORMAT('OWARNING - WATER LEVEL OF',F8.4,' MAY NOT CORRESPOND TO',
```

```
      1' GIVEN OUTFLOW OF',F8.4)
       GOTO 755
  745 CONTINUE
C   ***   CONVERGENCE ACHIEVED   ***
      H=HNEW
  755 CONTINUE
      IF (H.LT.HCRIT(IR-1)-HTOL2.OR.H.GT.HCRIT(IR)+HTOL2)
     1WRITE(6,975)H,IR
  975 FORMAT('OWARNING - WATER LEVEL OF',F8.4,' OUTSIDE RANGE',I4)
  765 CONTINUE
      HFROMQ=H
      RETURN
  795 CONTINUE
      WRITE(6,977)Q,H,FH,FDASHH
  977 FORMAT('OFAILURE - IN HFROMQ',4F8.4)
      STOP
      END
C
      SUBROUTINE ROUTE (P1,P2,R,H1,H2,DEL,IR)
C   ***   ROUTES INFLOW THROUGH RESERVOIR FOR ONE TIME STEP   ***
C   ***   EVALUATING H2   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      PBIG=0.5*DEL*(P1+P2)
      RBIG=R*DEL
      ITER=0
      H2=H1
  401 CONTINUE
C   ***   LOOP FOR NEWTON-RAPHSON ITERATIVE SOLUTION   ***
      QC=0.0
      QDASHC=0.0
      DO 410 ID=1,ND
      IF (IREF(IR,ID).EQ.0) GOTO 410
      IF (H1.LT.D(ID)+HTOL2) GOTO 405
      QC=QC+BC(ID)*(H1-D(ID))**E(ID)
  405 CONTINUE
      IF (H2.LT.D(ID)+HTOL2) GOTO 410
      QC=QC+BC(ID)*(H2-D(ID))**E(ID)
      QDASHC=QDASHC+BC(ID)*E(ID)*(H2-D(ID))**(E(ID)-1)
  410 CONTINUE
      FH=(H2-H1)*(A1*H2+A1*H1+2.0*A0)-2.0*(PBIG+AREA*RBIG)+DEL*QC
      FDASHH=2.0*(A1*H2+A0)+DEL*QDASHC
      IF (ABS(FDASHH).LT.HTOL2) GOTO 495
      H2NEW=H2-FH/FDASHH
      IF (ABS(H2NEW-H2).LT.HTOL) GOTO 425
      H2=H2NEW
      ITER=ITER+1
      IF (ITER.LT.ILIM) GOTO 401
C   ***   ITERATION LIMIT EXCEEDED   ***
      WRITE(6,949)P1,P2,R,H1,H2
  949 FORMAT('OCONVERGENCE FAILURE IN ROUTE',5F12.5)
      STOP
```

```
  425 CONTINUE
C   ***   CONVERGENCE ACHIEVED   ***
      H2=H2NEW
      Q2=QFROMH(H2)
      WRITE(3,947)IR,H1,H2,Q2,QC,QDASHC,RBIG,PBIG
  947 FORMAT(54X,I4,2F10.4,3F10.2,F10.4,F12.1)
      RETURN
  495 CONTINUE
      WRITE(6,948)H1,H2,FH,FDASHH
  948 FORMAT('0FAILURE IN ROUTE',4F8.4)
      STOP
      END
C
      INTEGER FUNCTION IRFRMQ (Q)
C   ***   FIND RANGE IN WHICH OUTFLOW LIES   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      DO 520 I=2,NR
      IR=I
      IF (Q.GE.QCRIT(IR-1).AND.Q.LT.QCRIT(IR)) GOTO 525
  520 CONTINUE
      WRITE(6,951)Q
  951 FORMAT('0WARNING - OUTFLOW OF',F8.4,' OUTSIDE RANGE')
  525 CONTINUE
      IRFRMQ=IR
      RETURN
      END
C
      INTEGER FUNCTION IRFRMH (H)
C   ***   FIND RANGE IN WHICH WATER LEVEL LIES   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      IR=1
      IF (H.LT.HCRIT(1)) GOTO 525
      DO 520 I=2,NR
      IR=I
      IF (H.GE.HCRIT(IR-1).AND.H.LT.HCRIT(IR)) GOTO 525
  520 CONTINUE
      WRITE(6,951)H
  951 FORMAT('0WARNING - WATER LEVEL OF',F8.4,' OUTSIDE RANGE')
  525 CONTINUE
      IRFRMH=IR
      RETURN
      END
C
      SUBROUTINE TRANS (P1,P2,R,H1,HT,DEL,T,IR)
C   ***   FINDS TIME AT WHICH WATER LEVEL REACHES SPECIFIED THRESHOLD   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
```

```
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      QSUM=0.0
      QQSUM=0.0
      DO 310 ID=1,ND
      IF (IREF(IR,ID).EQ.0) GOTO 310
      IF (H1.GT.D(ID)+HTOL2) QSUM=QSUM+BC(ID)*(H1-D(ID))**E(ID)
      IF (HT.GT.D(ID)+HTOL2) QQSUM=QQSUM+BC(ID)*(HT-D(ID))**E(ID)
  310 CONTINUE
      QBAR=0.5*(QSUM+QQSUM)
      X1=A0+0.5*A1*(H1+HT)
      X2=P1+AREA*R-QBAR
      XX=X2*X2-2.0*X1*(P2-P1)*(H1-HT)/DEL
      IF (ABS(P2-P1).LT.QTOL) GOTO 315
      T1=DEL*(-X2+SQRT(XX))/(P2-P1)
      T2=DEL*(-X2-SQRT(XX))/(P2-P1)
      GOTO 325
  315 CONTINUE
      T1=X1*(HT-H1)/X2
      T2=T1
  325 CONTINUE
      T=T1
      IF (T.GE.0.0.AND.T.LE.DEL) RETURN
      T=T2
      IF (T.GE.0.0.AND.T.LE.DEL) RETURN
      WRITE(6,939)P1,P2,R,H1,HT,DEL,T1,T2,IR
  939 FORMAT('0FAILURE IN TRANSITION'/8F12.4,I4)
      STOP
      END
C
      REAL FUNCTION HFROMD (DEPLET)
C   ***   CALCULATES WATER LEVEL FOR GIVEN DEPLETION   ***
      DIMENSION HMIN(20),BC(20),D(20),E(20)
      DIMENSION IREF(25,20)
      DIMENSION HCRIT(25),QCRIT(25)
      COMMON/RESCHA/HMIN,BC,D,E,
     1IREF,HCRIT,QCRIT,A0,A1,AREA,ALOW,HTOL,HTOL2,QTOL,DTOL,ILIM,ND,NR
      AA=0.5*A1
      BB=A0
      CC=DEPLET-HCRIT(1)*(A0+0.5*A1*HCRIT(1))
      DD=BB*BB-4.0*AA*CC
      IF (DD.LT.0.0) GOTO 351
      HFROMD=(SQRT(DD)-BB)/(2.0*AA)
      RETURN
  351 CONTINUE
      WRITE(6,931)
  931 FORMAT('0UNEXPECTED ERROR IN HFROMD')
      STOP
      END
```

```
C    ***   Test program for INTERP    ***
      DIMENSION QVAL(100)
      READ(5,*)NQVAL
      READ(5,*)(QVAL(I),I=1,NQVAL)
      CALL INTERP(QVAL,NQVAL,TPEAK,QPK)
      WRITE(6,961)QPK,TPEAK
  961 FORMAT(' Interpolated peak is',F8.3,' at',F6.2,'th value')
      STOP
      END
C
      SUBROUTINE INTERP(QVAL,NQVAL,TPEAK,QPK)
C
C    ***   To determine peak flow, QPK, and time of peak, TPEAK
C          by quadratic interpolation   ***
C
      REAL QVAL(NQVAL)
      INTEGER IPK,NQVAL
      QPK=QVAL(1)
      IPK=1
      DO 90 I=2,NQVAL
      IF (QVAL(I).LE.QPK) GOTO 90
      QPK=QVAL(I)
      IPK=I
   90 CONTINUE
      TPEAK=IPK
      IF (IPK.EQ.1.OR.IPK.EQ.NQVAL) RETURN
      Y=0.5*(QVAL(IPK-1)+QVAL(IPK+1))-QVAL(IPK)
      Z=0.5*(QVAL(IPK+1)-QVAL(IPK-1))
      IF (Y.EQ.0.0) RETURN
      TPEAK=IPK-0.5*(Z/Y)
      QPK=QPK-0.25*Z*Z/Y
      RETURN
      END
```

Appendix D Reservoir characteristics for case study catchments

Details of area-head and discharge-head relationships are given below for the reservoirs in the case study catchments. They appear in the same sequence as in the Tables in the main Report. Characteristics of the catchments draining into these reservoirs are given in Table 3.4: the areas shown in that table are inclusive of the reservoir lake area.

```
Description : Reservoired catchment example  1: Loch Craisg
Printed on 31  1 1992 at 11:18                   Run Reference : LOCHC
----------------------------------------------------------------------
Reservoir characteristics
**********************************************************************

   Reservoir area set to :      0.077 sq. km
                      at :      0.000 metres
        Area growth rate :      0.004 sq. km/metre

Device     HMIN        HMAX        BC          D          E

 1         0.000    9999.000     13.500      0.000      1.500

**********************************************************************
```

```
Description : Reservoired catchment example  2: Little Denny
Printed on 31  1 1992 at 11:19                   Run Reference : LITDY
----------------------------------------------------------------------
Reservoir characteristics
**********************************************************************

   Reservoir area set to :      0.120 sq. km
                      at :     96.650 metres
        Area growth rate :      0.008 sq. km/metre

Device     HMIN        HMAX        BC          D          E

 1        96.650    9999.000      7.950     96.650      1.500

**********************************************************************
```

```
Description : Reservoired catchment example  3: Loch Gleann
Printed on 31  1 1992 at 11:19                   Run Reference : GLEAN
----------------------------------------------------------------------
Reservoir characteristics
**********************************************************************

   Reservoir area set to :      0.138 sq. km
                      at :      0.000 metres
        Area growth rate :      0.007 sq. km/metre

Device     HMIN        HMAX        BC          D          E

 1         0.000    9999.000      4.680      0.000      1.500

**********************************************************************
```

Description : Reservoired catchment example 4: Parkhill House
Printed on 31 1 1992 at 11:20 Run Reference : PARKH

Reservoir characteristics

****

Reservoir area set to : 0.029 sq. km
at : 0.000 metres
Area growth rate : 0.001 sq. km/metre

Device	HMIN	HMAX	BC	D	E
1	0.000	0.610	1.309	0.000	1.500
1	0.610	9999.000	0.798	0.000	0.500

****

Description : Reservoired catchment example 5: Leperstone
Printed on 31 1 1992 at 11:20 Run Reference : LEPER

Reservoir characteristics

****

Reservoir area set to : 0.087 sq. km
at : 106.530 metres
Area growth rate : 0.004 sq. km/metre

Device	HMIN	HMAX	BC	D	E
1	106.530	107.030	6.325	106.530	1.674
1	107.030	9999.000	5.606	106.530	1.500

****

Description : Reservoired catchment example 6: Higher Naden
Printed on 31 1 1992 at 11:20 Run Reference : NADEN

Reservoir characteristics

****

Reservoir area set to : 0.052 sq. km
at : 0.000 metres
Area growth rate : 0.002 sq. km/metre

Level(metres)	Flow(cumecs)
0.000	0.000
0.100	0.600
0.200	1.600
0.300	3.000
0.400	4.600
0.500	6.400
0.600	8.400
0.700	10.800
0.800	13.300
0.900	16.200
1.000	19.400
1.100	22.600
1.200	26.100
1.400	33.900
1.600	42.700
1.800	52.000
2.200	72.000
2.800	105.000

****

Description : Reservoired catchment example 7: Lower Carriston
Printed on 31 1 1992 at 11:21 Run Reference : CARIS

Reservoir characteristics

Reservoir area set to : 0.097 sq. km
at : 94.180 metres
Area growth rate : 0.003 sq. km/metre

Device	HMIN	HMAX	BC	D	E
1	94.180	9999.000	17.710	94.180	1.500

Description : Reservoired catchment example 8: Nanpantan
Printed on 31 1 1992 at 11:21 Run Reference : NANPA

Reservoir characteristics

Reservoir area set to : 0.034 sq. km
at : 0.000 metres
Area growth rate : 0.002 sq. km/metre

Device	HMIN	HMAX	BC	D	E
1	0.000	9999.000	10.900	0.000	1.500

Description : Reservoired catchment example 9: Upper Neuadd
Printed on 31 1 1992 at 11:21 Run Reference : NEUAD

Reservoir characteristics

Reservoir area set to : 0.230 sq. km
at : 0.000 metres
Area growth rate : 0.008 sq. km/metre

Device	HMIN	HMAX	BC	D	E
1	0.000	9999.000	37.970	0.000	1.500

Description : Reservoired catchment example 10: Crafnant
Printed on 31 1 1992 at 11:22 Run Reference : CRAFN

Reservoir characteristics

Reservoir area set to : 0.216 sq. km
at : 182.430 metres
Area growth rate : 0.004 sq. km/metre

Device	HMIN	HMAX	BC	D	E
1	182.430	9999.900	3.283	182.430	1.510
2	183.340	9999.000	56.372	183.340	1.510

```
Description : Reservoired catchment example 11: Usk
Printed on 31  1 1992 at 11:22                     Run Reference : USKSN
------------------------------------------------------------------------
Reservoir characteristics
************************************************************************

   Reservoir area set to :      1.174 sq. km
                      at :    306.630 metres
        Area growth rate :      0.043 sq. km/metre

Level(metres)      Flow(cumecs)

     306.629            0.000
     306.934            5.857
     307.238           19.279
     307.543           38.993
     307.851           59.005
     308.458           97.246

************************************************************************
```

```
Description : Reservoired catchment example 12: Colt Crag
Printed on 31  1 1992 at 11:22                     Run Reference : COLTC
------------------------------------------------------------------------
Reservoir characteristics
************************************************************************

   Reservoir area set to :      0.850 sq. km
                      at :      0.000 metres
        Area growth rate :      0.100 sq. km/metre

Device     HMIN       HMAX        BC          D          E

 1        0.000      0.307     67.200      0.000      1.540
 1        0.307   9999.000     64.100      0.000      1.500

************************************************************************
```

```
Description : Reservoired catchment example 13: Loch Kirbister
Printed on 31  1 1992 at 11:23                     Run Reference : KIRBR
------------------------------------------------------------------------
Reservoir characteristics
************************************************************************

   Reservoir area set to :      1.015 sq. km
                      at :      0.000 metres
        Area growth rate :      0.051 sq. km/metre

Level(metres)      Flow(cumecs)

       0.000            0.000
       0.076            0.170
       0.168            6.230
       0.305           19.610

************************************************************************
```

```
Description : Reservoired catchment example 14: Staunton Harold
Printed on 31  1 1992 at 11:23                    Run Reference : STAUN
--------------------------------------------------------------------
Reservoir characteristics
********************************************************************

   Reservoir area set to :      0.880 sq. km
                      at :      0.000 metres
        Area growth rate :      0.035 sq. km/metre

Level(metres)        Flow(cumecs)

       0.000               0.000
       0.305              14.160
       0.610              35.400
       0.762              48.140
       0.914              65.130
       1.067              85.000
       1.219             107.600
       1.372             126.300
       1.524             132.500

********************************************************************
```

```
Description : Reservoired catchment example 15: Roadford
Printed on 31  1 1992 at 11:23                    Run Reference : ROADF
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Reservoir characteristics
********************************************************************

   Reservoir area set to :      2.960 sq. km
                      at :    126.400 metres
        Area growth rate :      0.172 sq. km/metre

Level(metres)        Flow(cumecs)

     126.400               0.000
     126.600               9.500
     126.800              22.300
     127.000              39.500
     127.200              61.500
     127.400              88.000
     127.600             117.000
     127.800             150.500
     128.000             185.000
     128.100             200.000

********************************************************************
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