Flood Studies Supplementary Reports

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Flood Studies Supplementary Report No 1 Sept 1977

The Areal Reduction Factor in rainfall frequency estimation

1. INTRODUCTION

The Flood Studies Report (FSR) definition and use of Areal Reduction Factor (ARF) has aroused some interest among engineers and hydrologists. There has also been considerable misunderstanding. This note is intended to amplify the account given in FSR Vol II Chapter 5 and to summarise the results of a subsequent and more specific investigation designed to test the suitability of the recommended values.

It is concluded that the FSR values of ARF *are* appropriate for use in current design because:

(a) there is no evidence for geographical variation
(b) although there is a tendency for ARF values to decrease with increasing return period, this may be neglected for practical purposes because such variations are small compared to the effects of other simplifying assumptions.

Despite the belief that FSR users can safely continue to apply the published ARF values, it is possible that, as longer records of short duration rainfall become available in the future, some revision of ARFs for higher return periods may be justified.

2. WHAT IS ARF?

Very great care must be taken to distinguish between two quite different definitions of ARF.

In the FSR and in this note we are concerned with the factor which relates the statistics of point rainfall to those of areal rainfall thus:

$$Ra = ARF \times Rp$$

where, for a given duration and return period, Ra and Rp are the expected rainfall depths over an area and at a point (actually the mean of all point values within the area) respectively.

It is this definition that concerns the engineer who designs against an event on a catchment *area* but must use rainfall statistics based directly on raingauge records collected at *points*.

The second definition is the storm-centred ARF which describes the way in which rainfall intensity decreases with distance from the centre of the storm in individual events.

In estimating maximum floods (with 'infinite' return period) some designers may wish to look at storm-centred ARFs but, even in this case, the use of the 'statistical' ARF may still be preferred because it is usually the larger and hence more conservative number.

When the distinction between the two definitions is clearly understood it becomes obvious that comparisons are invalid; the statistical ARF cannot be discussed in the context of individual storm characteristics because point statistics do not refer to storm maxima.

3. DERIVATION OF ARF

The best way of deriving the statistical ARF is:

(a) for a selected duration and area, produce frequency curves for rainfall stations in the prescribed area and take an average curve to represent a typical point in the area.

(b) for the same duration and area calculate annual maximum values of the average areal rainfall (by isohyets or by simple or weighted averaging) and produce a frequency curve.

(c) ARF, at various return periods, is simply the ratio of ordinates between the two curves (Figure 1)

(d) repeat for other durations and areas in various regions.

This procedure is easy to describe but extremely laborious to undertake. It could not be attempted within the time available for producing the FSR and an indirect approach was used.

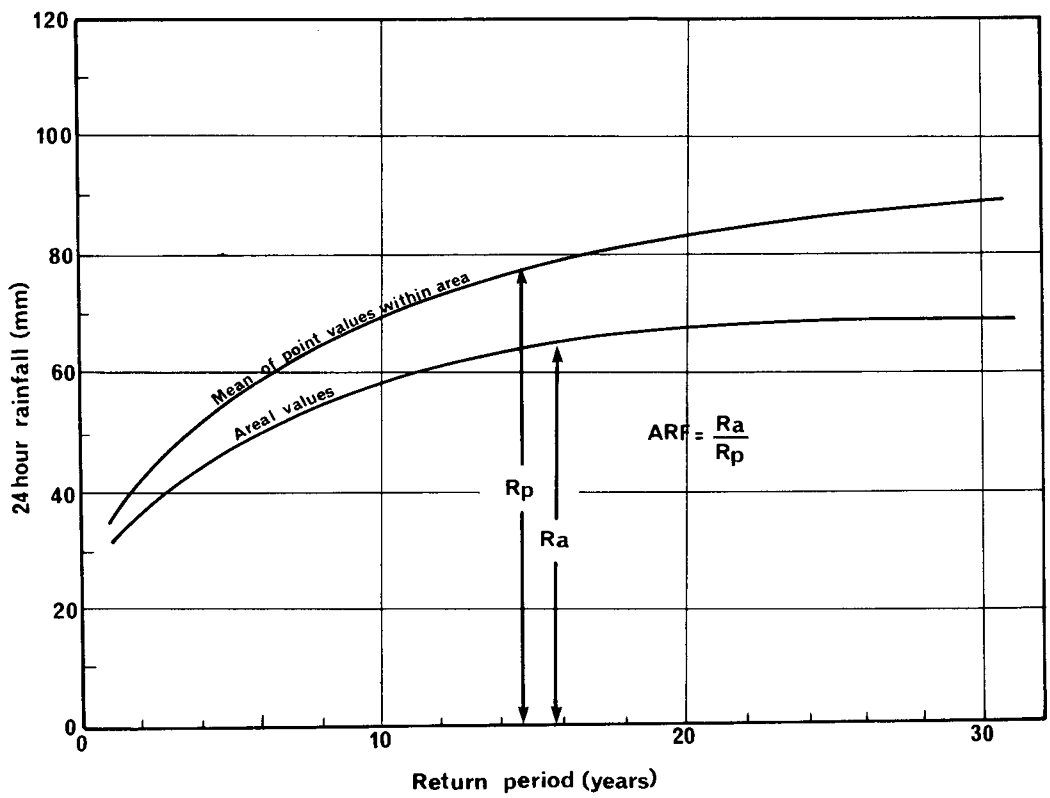

FIGURE 1 Interpretation of Areal Reduction Factor

In the FSR, two assumptions are implicit in the procedure used. The first is that ARF does not vary with return period. The second is that 'an average of ratios' could safely approximate 'a ratio of averages'.

The second assumption deserves a little more explanation. In a given area there are M gauges each with N years of record. For the jth gauge the highest rainfall in a given duration in the ith year of record is $R1_{i,j}$. The mean value for all years and all stations is

$$\overline{R1} = \frac{1}{M \times N} \sum_{i=1}^{N} \sum_{j=1}^{M} R1_{i,j}$$

The highest <u>areal</u> rainfall in each year may be estimated as

$$\frac{1}{M} \sum_{j=1}^{M} R2_j$$ (where R2 is a <u>point</u> rainfall contributing to an *areal* maximum and will often be the same as R1.)

and the average value in the N year period is

$$\overline{R2} = \frac{1}{M \times N} \sum_{i=1}^{N} \sum_{j=1}^{M} R2_{i,j}$$

As any variation with return period is being ignored, the correct estimate of ARF is

$$ARF = \overline{R2}/\overline{R1}$$ (this is the 'ratio of averages')

Page 39 in Volume II of the FSR explains that R2/R1 was calculated at each station and for each year before averaging. Thus

$$ARF = \frac{1}{M \times N} \sum_{i=1}^{N} \sum_{j=1}^{M} (R2/R1)_{i,j} = \overline{(R2/R1)}$$ (this is the average of ratios)

Now that more time is available, the Institute and the Meteorological Office aim to check the validity of the first assumption (ARF independent of return period) and to consider the effects of both assumptions on the accuracy of the values obtained and on the conclusion that they did not vary with location. To this end, the Institute invited F. C. Bell (on sabbatical leave from the University of New South Wales) to do some work involving derivation in the preferred manner described at the start of this section. His study is reported in detail in IH Report No. 35 available free from the Institute. The remaining sections of this note draw on the main conclusions from Bell's work.

4. DOES ARF VARY WITH RETURN PERIOD?

Bell chose nine areas each of 1000 km^2 containing at least 12 raingauges with 14 years of record. He derived point and areal rainfall frequency curves for daily rainfall. He also used a few smaller (20 and 100 km^2) and larger (8000 km^2) areas and studied 1 and 2 hour rainfall frequencies. Figure 2 shows the location of all the areas.

1 Scottish Highlands
2 Aberdeen
3 Newcastle-Hexham
4 Belfast
5 Norwich
6 Plymouth
7 Grendon Underwood
8 Plynlimon
9 River Dee
10 Surrey
11 Surrey
12 Chilterns
13 Greenwich
14 Grendon Underwood
15 Plynlimon

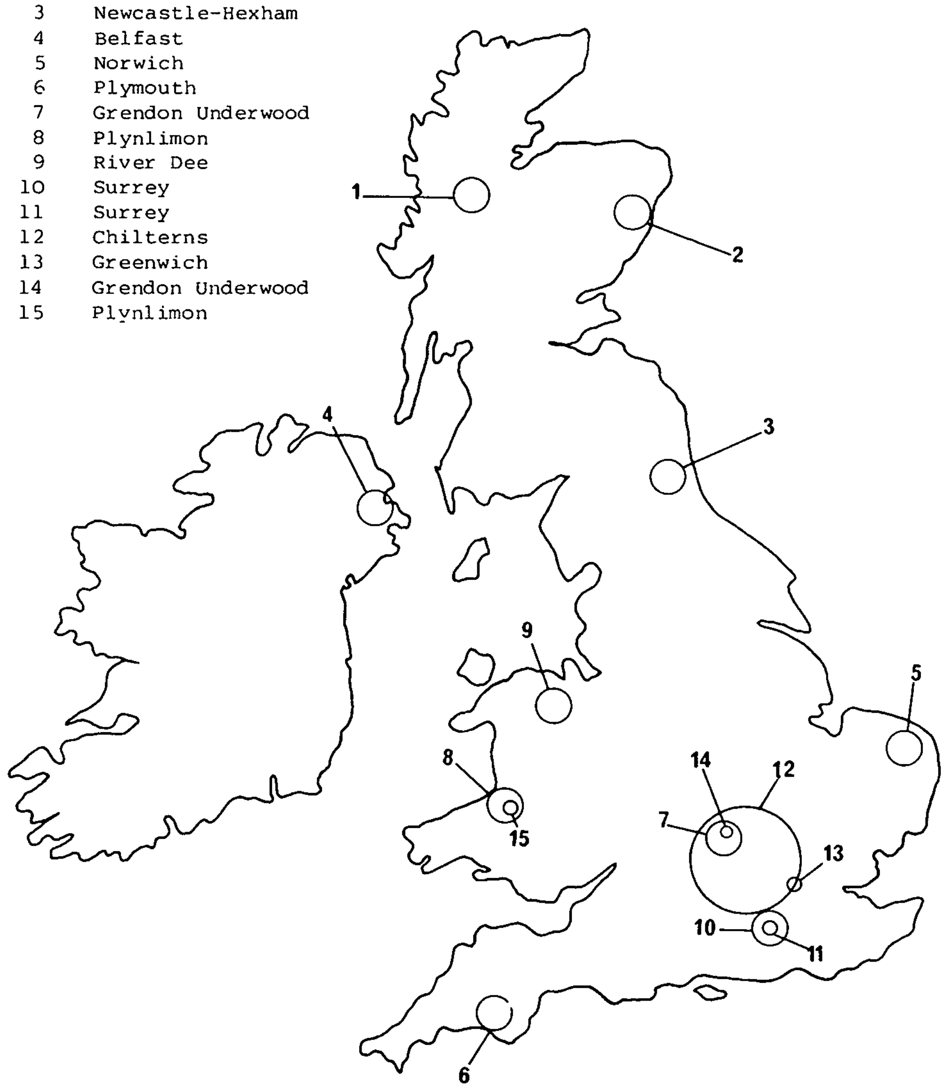

FIGURE 2 Location of areas selected for analysis of areal rainfall

The data provided reasonable evidence that ARF decreases with increasing return period (Table 1) but it is thought that, until longer records are available to test the influence of the assumed distribution type (exponential) on the magnitude of the effect, the conservative recommendation that ARF be assumed independent of return period should stand.

SAMPLE AREA	2-YEAR R.P.		5-YEAR R.P.		10-YEAR R.P.		20-YEAR R.P.	
	ARF	S.E.	ARF	S.E.	ARF	S.E.	ARF	S.E.
1	.95	.04	.93	.06	.92	.08	.91	.10
2	.95	.04	.92	.05	.90	.07	.89	.10
3	.89	.04	.89	.06	.89	.08	.89	.09
4	.90	.06	.90	.08	.90	.10	.90	.13
5	.95	.05	.88	.05	.86	.07	.82	.09
6	.90	.04	.88	.07	.86	.09	.85	.11
7	.93	.04	.87	.06	.86	.08	.82	.10
8	.86	.04	.85	.06	.84	.08	.84	.11
9	.89	.04	.84	.05	.81	.05	.80	.05
MEAN	.91	*.04	.88	*.06	.87	*.08	.86	*.10
EXPECTED ARF FROM F.S.R. = .89 FOR ALL SAMPLE AREAS								

*mean S.E. was calculated from: $\sqrt{\frac{\Sigma S.E.^2}{9}}$

S.E. = Standard error

Table 1. ARF calculated from areal and average point rainfalls having the same return period

5. WHAT EFFECT DOES THE DERIVATION METHOD HAVE ON ARF VALUES?

It was shown earlier that the FSR method of deriving ARF, adopted for computational convenience, is slightly unorthodox. Although the more rigorous procedure adopted by Bell is to be preferred for future use, it is reassuring that the values obtained are sensibly the same.

6. DOES ARF VARY WITH LOCATION?

The differences between the nine 1000 km^2 areas in Table 1 are within the error of estimating ARF. There might be a slight trend for an increase with latitutde but the effect is of no practical significance. The FSR conclusion that ARF is independent of location is therefore supported by Bell although he widened the range of climate by studying areas in Scotland and Wales.

Flood Studies Supplementary Report No 2

The estimation of low return period floods

1. INTRODUCTION

There are circumstances when knowledge of the size of low return period floods is important, in particular in cost-benefit analysis of flood protection when at present costs are incurred even under frequent flooding conditions.

Analysis of annual maximum data yields return periods which are the *mean interval between years* containing a flood above a given size. By definition this return period cannot be less than one year; the smallest flood in a very long series of annual maxima would have a return period of about one year. A peaks over a threshold (POT) return period, found by analysing all floods above a given size, however, is the *mean interval between floods* exceeding the given size and there is no restriction to how small that interval can be other than the time which must elapse between two peak flows in order to call them separate floods.

In most cases where costs due to frequent flooding are to be allowed for, the POT type of return period is most appropriate as for example with property damage or transport disruption. In special circumstances where it is not possible for damage to occur more than once in each year then the annual maximum return period is the correct one to use. Such circumstances might be crop damage after which it is not possible to resow until the following season.

2. PRESENT RECOMMENDATIONS

If one wished to know, for example, what the twice a year flood was at an ungauged site using the FSR one would proceed as follows.

(a) Using chapter I.4 obtain an estimate of the mean annual flood from catchment characteristics
(b) Using the region growth curves of section I.2.6.2 produce a flood frequency curve down to a return period of about 1.1 years. One would need to use the equation fitted to the region curve to do this (Table I.2.38).
(c) Obtain from section I.2.2.3, figure 2.3 the return period (1.16 years) on the annual maximum series corresponding to a return period of 0.5 years on the POT series.
(d) From the flood frequency curve produced in (b) read off the flood value at 1.16 years and this is the twice a year flood.

3. REVISED RECOMMENDATIONS

The procedure above is based on a theoretical relationship between the return periods of POT series and annual maximum series deduced by Langbein.

$$T_{AM} = \left[1 - \exp\left(- \frac{1}{T_{POT}} \right)\right]^{-1}$$

where T_{AM} = return period of a given flood on an annual maximum series.

T_{POT} = return period of the same flood on a peaks over a threshold series.

Analysis of data for Great Britain shows some depatures from this theoretical relationship and Table 1 is based on this data to enable growth factors to be read off directly for small return periods.

The new procedure is to estimate the mean annual flood as in (a) and to read from table 1 the growth factor for the required return period. The required flood is then the product of the mean annual flood and the growth factor.

FSR Growth Curve Region	Return Period (years) 0.2	0.5	1.0	2.0	5.0
1	0.57	0.72	0.85	1.01	1.24
2	0.65	0.76	0.87	1.00	1.21
3	0.48	0.68	0.86	1.04	1.27
4	0.50	0.67	0.83	1.00	1.27
5	0.56	0.72	0.87	1.05	1.34
6) 7)	0.51	0.69	0.85	1.04	1.33
8	0.38	0.61	0.78	0.99	1.27
9	0.57	0.73	0.88	1.03	1.24
10	0.58	0.74	0.87	1.02	1.22

Table 1. Regional values of $\frac{Q}{\bar{Q}}$ corresponding to various return periods

4. BACKGROUND TO THE REVISION

The relationship between the two sets of return period was examined on the data from 40 long term gauging stations throughout Great Britain. It was found that the relationship between T_{AM} and T_{POT} varied across the country. Although this variation was not readily attributable to the type of catchment it was possible to identify four groups in which the relationship was fairly constant. The make-up of these groups in terms of the FSR growth curve regions is as follows.

	Growth curve Regions
Group A	3, 8
Group B	4, 10
Group C	122, 9
Group D	5, 6, 7

Table 2 shows the average relationship between the two types of return period for the four groups along with the theoretical relationship.

T_{POT} Years	Group A	B	C	D	Theoretical
0.2	1.01	1.02	1.03	1.06	1.01
0.5	1.19	1.22	1.25	1.36	1.16
1.0	1.64	1.69	1.74	1.90	1.58
1.4	2.02	2.08	2.14	2.31	1.96
2.0	2.61	2.67	2.72	2.89	2.54
5.0	5.48	5.52	5.51	5.53	5.52

Table 2. Annual Maximum return periods for various POT return periods.

The greatest departures from the theoretical values (and hence the present recommendations) occur in group D, the south east of England. Table 1 has been derived from Table 2 and the region growth curves of Section I.2.6.2.

A fuller description of this study has been published by Beran and Nozdryn-Plotnicki in the Bulletin of the International Association of Hydrological Sciences, 1977, 2 (2), 275-282.

5. EXAMPLE

The twice a year flood is required on the river Rother just upstream of Chesterfield. The mean annual flood from catchment characteristics has been calculated as 21.4 cumecs. The Rother is in hydrometric area 27 and in FSR growth curve region 3, so the growth factor for the twice a year flood (return period 0.5 years) is 0.68 from Table 1. Hence the twice a year flood is 0.68 x 21.4 = 14.6 cumecs.

Flood Studies Supplementary Report No 3 Oct 1977

A report of the seminar

The Flood Studies Report - an opportunity for discussion

Birmingham University, 24-25 March 1977

1. INTRODUCTION

The two-day seminar to discuss the Flood Studies Report (FSR) was organised by the University of Birmingham, Department of Civil Engineering, in association with the Institute of Hydrology. The seminar provided an opportunity for users and critics of the FSR to come together and talk about the shortcomings and problems of using the report. The sessions were chaired by Professors M. J. Hamlin (Birmingham University) and T. O'Donnell (Lancaster University). After discussion of each of the points raised the chairmen attempted to classify them into one of the following categories.

Category 1. The comment is valid but does not have sufficient applicability to warrant a change in the recommendations of the FSR.

Category 2. The criticism is valid and the FSR should be changed to take this into account.

Category 3. The point touches on a topic where insufficient is known and further research is required before any changes to the FSR would be warranted.

A great many points were raised and those which were answered satisfactorily by the FSR authors are not dealt with in this report. The rest of this report is taken from the Chairmen's comments which have already been distributed to the seminar participants.

2. Chairmen's Introductory Comment

We feel it necessary to emphasise the comments made at the start of the seminar, viz. that the basic purpose of the FSR is to provide design techniques for predictions about flood magnitude/frequency relationships; that those techniques must be equally applicable in the "no data" situation and the "data available" situation; that the FSR is not concerned with flood forecasting nor with physically well-founded catchment modelling. It seemed to us that criticism of the FSR not infrequently lost sight of one or other of these constraints.

The following comments fall under the session headings used during the seminar.

3. Data Base

Users have found catchment parameter values that lie outside the ranges of values used in preparing the FSR. Of the parameters mentioned at the seminar, we feel that both slope and area parameter values out of range to the extent reported can be tolerated because of the multi-dimensional nature of the equation (Category 1). We are, however, concerned that stream frequencies have been found some 5 times larger than the maximum on any gauged catchment used in the FSR.

Enmeshed in this problem are the questions of OS delineation of streams (consistency between surveyors), and discrepancies between First Series and Second Series maps, factors discussed again in the closing session. We consider this to be a Category 3 problem.

The need for peak stage as well as peak flow rate as a flood index was proposed. Regionalised flood level/frequency maps have been prepared in the USA and similar work could be done for the UK. We feel that this is a low priority Category 3 matter.

The advantages (and the dangers) of a "slim guide" to the FSR techniques were debated. We recommend the speedy publication of the draft IH slim guide (liberally salted with caveats); it is important to prevent a Babel-like multiplicity of such guides from RWA's and others.

4. Mean Annual Flood Estimation (the "no data" case)

Criticism was voiced on the FSR recommendation for use of the parameter RSMD (rather than SAAR) since it was tedious to evaluate, and the regressions using RSMD were not much better than those using SAAR. Since publication of the FSR a map of RSMD values has been prepared and was published in the Proceedings of the I.C.E. Conference on Flood Studies held in May 1975. This RSMD map should be sent with the appropriate documentation to purchasers of the FSR (a Category 2 matter).

Several aspects of the validity of the FSR regression equations for $\bar{Q}$ were discussed. We concur with the FSR conclusion that for a general purpose omnibus equation applicable to all types of catchment, the 6-variable equation recommended is appropriate. A designer is, of course, free to use a regression with fewer variables - he is fully supplied in the FSR with measures of the increased errors of estimate resulting. We view the seminar comments on this issue of how many variables to use as falling into Category 1.

Other aspects of the $\bar{Q}$ regression equations cause us more concern. The most pressing point, a Category 2 one, is that of the regional multipliers for Northern Ireland. The values quoted at the seminar and based upon recently collected data should be verified and corrections published, and the effect on the Republic of Ireland multipliers (of deleting the N. Ireland catchments) examined. Arising from this, as a Category 3 point, is the periodic re-evaluation of all the regional multipliers as more catchment records become available.

The dilemma of what multiplier to use for catchments at the boundary between two or more regions needs to be resolved. Perhaps comparisons should be made between the means of the existing multipliers for any two adjacent regions and new multipliers obtained for groups of catchments surrounding each common boundary. We consider this a high priority Category 3 question.

There was considerable comment on the omission of a floodplain/washland storage parameter from the $\bar{Q}$ regressions. Recognising that catchments differing only in such storage characteristics could be expected to have different $\bar{Q}$ values due to different degrees of storage attenuation, we consider that there may be a weakness that needs examination (as a Category 3 matter). It is not easy to take a good index of floodplain storage from a 1 : 25000 map (25 ft contour intervals), yet such a constraint exists if one is to make predictions for an ungauged catchment. A rough yet possibly adequate enough index of storage might arise from an average measure of transverse floodplain slope towards the river channel. This in turn could be indicated by averaging the angles made by the 25 ft contour lines as they meet the river channel. We recommend a pilot study to test the usefulness of such a parameter in the regression equations.

The difficult issue of how to allow for improved land drainage must, we feel, await the results of research being conducted in the Irish Republic, perhaps paralleled by similar work in the UK (Category 3). A subjective stop-gap procedure might be to enhance the soil index parameter to allow for higher peaks and greater volumes; however, this will not result in more rapid post-works drainage.

As a postscript to all the above comments, a sense of proportion must be exercised in judging the $\bar{Q}$ regression equations. Users of the FSR should read and heed the cautionary advice on pp. 342 ff and temper their views of the reliability of the regression estimates of $\bar{Q}$ accordingly. The FSR does not by-pass engineering judgement. As a particular example, the question of the effects of floodplain storage on $\bar{Q}$ might well have been answered along the lines that judgement should be exercised by first using the present regression equations to find $\bar{Q}$, and then, if the catchment has excessive floodplain storage, assessing the reduction in $\bar{Q}$.

5. Regional Growth Curves

There was some comment on the distributions to be used in the fitting of annual peak flows and in particular on the problem of standard errors of estimate apparently reducing with larger return periods using the 3-parameter GEV distribution when $k > 0$. In accepting the validity of the comment, the Flood Studies team reiterated their view in the Report that the GEV-based equation I.4.3.13 was not to be used for standard error calculation; Table 2.26, and Table 2.37 using the EV1 distribution, give guidance on lower bounds for standard errors to be used for all cases. However they also implied that preference should be given to the two parameter distribution; if this represents their current view it is a Category 2 matter.

The above problem was linked to values of regional skewness. A user argued that skewness values were not related to regional characteristics, but to catchment characteristics. Thus, for example, the presence or absence of a flood plain upstream of a gauging site was much more likely to affect the skewness of the distribution of annual flood peaks than any inherent difference between one region and another. However, this seems to us to be a Category 3 topic.

The regional growth curves were discussed at length, with individual comments on the apparently inconsistent discrepancies between various regions and particularly between adjacent regions, where the boundary catchment dilemma discussed already for the $\bar{Q}$ regression equations again appears. A similar high priority Category 3 study comparing means of adjacent regional u, α and k values in Table 2.38 with new values for catchment groups surrounding a boundary would illuminate this dilemma.

The basis on which the regional growth curves were derived appears to us to be sound, the use of other possible subdivisions of the data by AREA, S1085, SAAR etc having been adequately investigated and found wanting. It is not, perhaps, appreciated sufficiently that one of the reasons for the different curvatures in the growth curves, which are, after all, functions of the regional $\bar{Q}$, arises simply because the value of $\bar{Q}$ for a catchment of a given size varies considerably from one region to another.

6. Rainfall Statistics

Under this heading, three important matters were discussed. The first of these was the discrepancy between the return intervals of storms which were considered to be fairly common and the very long return intervals attributed to such storms using the relationship developed by the Meteorological Office and given in

Volume II. Whilst the quantity of data analysed for Volume II was very considerable and the results should represent a significant improvement on the earlier analysis by Bilham, there nevertheless appears to us to be a very real need for the Meteorological Office to examine and report on the discrepancies which were raised (Category 2). Specific M(T)/M(5) values for Somerset were quoted which were inconsistent with the rainfall growth curve in that area.

There seems to us to be some users' misinterpretation of both the derivation and the purpose of the storm profile developed in Volume II. The profiles are averages of the concentrations of rainfall within sub-periods of the storms, and as used in the FSR represent an idealised design storm. It is inevitable that different recorded storm profiles will produce different run-off patterns, and that there is an infinite number of such different distributions of rainfall possible within a storm of a given length. However, the FSR is not concerned with flood forecasting from a particular rainfall event. (Category 1). It is perhaps appropriate to re-iterate the point made earlier that the basic purpose of the FSR is to provide design techniques for predictions about flood magnitude/frequency relationships.

The third topic on rainfall concerned the areal reduction factors. We have little doubt that the ARFs are badly explained in the FSR, and consequently have been misunderstood, and mis-used. A clarification of this matter is required for all purchasers of the Report. It also seems to us important that the results of Bell's later work at the Institute on ARF's should be incorporated into this clarification. Together, these form a high priority Category 2 matter.

7. Rainfall/Runoff Model

This topic produced a number of contributions including comments on the shape of the unit hydrograph and the Q_p T_p value. In our view all these points were dealt with satisfactorily in the discussion, including the design use of the model in establishing peak flows of specified return periods.

8. EMP/EMF

In dealing with extreme events there was some confusion as to whether the upper limit of the Volume II growth curves should be used or whether the use of EMP maps and the FSR method was to be preferred. It was stated that the EMP maps/FSR method was considered to be superior. If so, this clarification is in our opinion very important and should be transmitted to all purchasers of the Report (Category 2, high priority).

9. Urbanising Catchments

There were a number of contributions dealing with urban drainage. Our main conclusion is to urge the need for early publication of the work currently being undertaken in association with the CIRIA and DOE working parties (Category 3).

However it was made clear that the present FSR methods based upon the rainfall runoff model (a) should be limited to catchments where the urban fraction did not exceed 25% and (b) ought not to be used to predict the effects of increasing urbanisation.

10. Miscellaneous

The comparison between different methods of determining the value of $\bar{Q}$ presented by one of the contributors demonstrates again the need for the designer to base his ultimate choice of flood value on his own experience and engineering judgement. It is unrealistic to expect the answer from each of the methods set out in the FSR to be the same for any one catchment out of the whole range of British catchments with very different topographies and soil types. This topic is regarded as being Category 1.

There is a need to update the soil type maps, particularly in small upland catchments (Category 3).

The need to "look again" with an increased databank is acknowledged in the Report, and we now understand is covered in part by a Ministry of Agriculture contract with IH, but within three or four years a decision on how to finance this additional work will have to be taken. It is expected that little of the remaining unexplained variance will be taken up although the work on regional multipliers and regional growth curve parameters might benefit from the increased availability of data. We also feel that the studies proposed earlier of inter-region boundary values of multipliers and growth curve parameters could well lead on eventually to a contouring presentation of these quantities (Category 3).

The suggestion that there should be an additional loose-leaf Volume VI for errata and new or revised information, to be sent to all purchasers of the Report, has already been taken up by the Institute in the form of a series of Supplementary Reports.

We strongly endorse the suggestion that the collection and publication (in Volume VI?) of a number of users' worked examples would be an extremely valuable addition to the worked examples in the Report and the proposed "slim guide". This would enable the Institute to monitor the use of the Report and would enable the user to assess better the results of his work.

SUMMARY OF THE MAIN POINTS RAISED AT THE SEMINAR

Category 1

Catchment slope and area parameter values outside range of data used in correlation.

Number of variables in regression equations.

Storm profiles.

Variation of $\bar{Q}$ determined by different methods.

Category 2

Issuing of RSMD maps.

Regional multiplier for Northern Ireland; Republic of Ireland multiplier.

Advice on use of 2-parameter as against 3-parameter distributions.

Answer by Meteorological Office of the points raised on rainfall statistics.

Clarification of ARFs and publication of work by Bell (high priority).

The recommended use of the EMP Maps c.f. M(T)/M5 growth curves (high priority).

Category 3

Catchment stream frequency parameter values outside range of data used in correlation.

The effects of differences between Series 1 and Series 2 maps.

Regionalised flood level/frequency maps (low priority).

Periodic re-evaluation of regional multipliers, and regional growth curve parameters.

The examination of multipliers and growth curve parameters along boundaries of regions (high priority).

Flood Studies Supplementary Report No 4 Nov 1977

Some results of a search for historical information on chalk catchments

1. INTRODUCTION

Observations on the statistical behaviour of floods from chalk streams show two main characteristics: the magnitude of the peak discharge per unit area is much lower for chalk streams than for other rivers; and the year to year variability of peak discharges is somewhat less than other streams in the same parts of the country.

However, engineers with experience of chalk streams often refer to the occasional large peak, many times greater than those normally observed. The only notable examples among the recorded data given in the Flood Studies Report (FSR) are for the Lud at Louth in Lincolnshire which in 1920 produced a flood 31 times the median annual flood, and the Whitewater at Lodge Farm, a Thames tributary, where in March 1947 a flood peak of over 10 times the median was experienced. However, neither of these refer to events recorded during the period of conventional record but relate to special estimates made at the time. A historical search was undertaken to try to find further evidence of such events and hopefully improve the definition of the flood frequency relationship appropriate to chalk streams.

The results of the past statistical analyses are reviewed in the next section. Section 3 discusses the findings of the historical search which tends to support the evidence of the discharge data for medium and large catchments but points to the real likelihood of high runoffs from headwater areas.

2. STATISTICAL ANALYSES OF RUNOFF DATA

There is a large literature on chalk hydrology but as might be anticipated from its importance to water supplies much of it relates to yield from groundwater. Attempts to produce relationships between the mean annual flood (average of annual maximum flows) and catchment characteristics are less successful where chalk catchments are prominent than for other types in that the error about the regression line is large and the inclusion of catchment characteristics is not clear-cut (FSR, pp I.335 and I.344).

Table 1 shows the specific mean annual flood for a number of catchments consisting entirely of chalk. The runoff values are very low by comparison with other rivers in the same region; values ten times greater are frequently found. The lowest values are found for catchments with a large proportion of Upper Chalk. Data are only available from perennial streams so the list does not include data from small headwater tributaries.

Table 2 shows the flood frequency relationship obtained by combining the data from the Table 1 station's annual maximum data using the method of FSR, I.2.6.3. This relationship is considerably flatter than for other catchments in the same region indicating low year to year variability in the maxima.

Table 1 could be used to check the mean annual flood value that is obtained from a short record or from a regression equation. Due attention should be paid to the nature of the chalk in making comparisons. Table 2 is an example of the injunction in the FSR to form region curves for specific purposes if there is reason to suppose that the data set is coherent.

Table 1. Specific flood runoffs for chalk streams

	Area, km^2	Hydrometric region	Specific MAF, m^3/s per km^2
Kennet at Marlborough	142	39	0.021
Kennet at Theale	103.4	39	0.040
Lambourn at Shaw	234	39	0.015
Lambourn at Welford	176.1	39	0.011
Lambourn at East Shefford	119	39	0.014
Winterbourne at Bagnor	49.2	39	0.007
Meon at Mislingford	72.8	42	0.041
Avon at Amesbury	324	43	0.037
West Beck at Wanford B	57.2	26	0.035
Waithe Beck at Brigsley	108	29	0.014
Stringside at White Bridge	93.5	33	0.027
Beechamwell Brook at Beechamwell	34.4	33	0.012
Mel at Meldreth	8.6	33	0.045
Ant at Honing Lock	49.3	34	0.019
Whitewater at Lodge Farm	44.6	39	0.025
Itchen at Allbrook	360	42	0.026
Avon at Ringwood	1640	43	0.039
Law Brook at Albury	16.2	39	0.021

Table 2. Flood magnitude-frequency relationships

Return period, years	Chalk streams $Q(T)/\bar{Q}$
2	0.92
5	1.26
10	1.48
25	1.77
50	1.98

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3. RESULTS OF THE HISTORICAL SEARCH

This aspect of the study was carried out by H R Potter and entailed a 6-month programme of enquiry into archival material at all the main centres in the chalk belt. A very large quantity of material relating to heavy rainfall and consequent flood events on chalk streams plus an even larger amount on bourne flow was uncovered (See Appendix A).

However it has to be admitted that very little of this material enabled any improvement to be made to the statistics describing flood flows. The overriding reason was that the mass of the information related to headwaters and dry valleys which are ungauged. Some references described events in which the soil was eroded and scars remain to this day. Such references seemed prevalent in East Yorkshire but accounts also appear in the Chilterns. A reconnoitre using aerial photographs showed similar erosion scars in every part of the chalk belt.

Thus the conclusion of this search must be that the larger chalk streams such as appear in the gauged data sets are adequately described by the present recommendations supplemented by the information given in the previous Section. However, it would appear perfectly possible for small headwater areas and dry valleys, commonly of the order of a few hectares in area, to receive intense storms and shed the runoff at a rate comparable with catchments on less permeable soils. This type of event has seldom given rise to flooding of habitation.

The descriptive material has been collected together and catalogued so that it is available for inspection at IH. Alternatively, individual items from Appendix A can be requested.

APPENDIX A

This appendix lists the main results from the historical search for chalk flooding instances. In each case a form is available giving location and event date or dates. Additionally the event is briefly described and the reference quoted which gives further details. Any numerical information e.g. antecedent rainfall and levels if available are also quoted. This sheet plus a copy of the reference, if available, can be supplied on request.

HYDROMETRIC AREA	LOCATION	RIVER	DATE
26	Towthorpe	Hull (Trib.)	1888
26	Towthorpe	Hull (Trib.)	1894
26	Langtoft	Hull (Trib.)	1657
26	Langtoft	Hull (Trib.)	1853
26	Langtoft	Hull (Trib.)	1888
26	Langtoft	Hull (Trib.)	1892
26	Huggate	Hull (Trib.)	1892
26	Great Driffield	Hull (Trib.)	1892
26	Great Driffield	Hull (Trib.)	1910
26	Weaver Thorpe	Gypsey Race	1910
26/27	South Cave	Foulness/Mkt Weighton Canal	1912
26/27	Market Weighton area	Market Weighton Beck	1973
27	East Heslerton	Derwent (Trib.)	1910
29	Louth	Lud	1920
29	Louth area	Lud	1149-1315
29	S Elkington and N Ormsby	Lud (Trib.)	1893
29	Wold Newton	Waithe Beck (Trib.)	1875
30	Horncastle	Bain	1960
33/38	Royston and Watford Area	Gt Ouse/Lea	1900
33/39	Wendover area	Gt Ouse/Thame	1950
34	Norwich	Yare	1912
34	Norwich	Yare	1619-1912
38	Hemel Hempstead	Lea (Trib.)	1865
38	Hatfield	Lea (Trib.)	1865
38	Dunstable area	Lea (Trib.)	1938
39	Marlborough	Kennet	1895
39	Chilterns area	Missbourne/Chess	1918
39	Chilterns area	Wey & Thames Tribs.	1936
39	Watford	Colne	1901
39	Epsom Downs	Mole	1910
39	Banstead	Mole	1911
39	Benson	Thames	1920
41	Steyning	Adur	1872
41	Lewes	Ouse	1788
41	Lewes	Ouse/Winterbourne	1960
42	Selbourne	Tribs.	1784
43	Salisbury	Avon	1092-1960
44	Cerne Abbas	Cerne	1889
44	Winterbourne Steepleton	S Winterbourne	1955
44	Uplyme	Lymm	1886
44	Dewlish	Dewlish Brook	Prehistoric

Flood Studies Supplementary Report No 5 Sept 1979

Design flood estimation in catchments subject to urbanisation

1. INTRODUCTION AND SUMMARY

Urbanisation can have a dramatic effect on the flood response of a catchment, both in terms of hydrograph shape and in terms of the flood magnitude-frequency distribution. Complete urbanisation typically reduces hydrograph rise time by 75% and increases mean annual flood by between 200 and 600% depending on the responsiveness of the catchment before urbanisation. These effects were not specifically investigated in the Flood Studies Report (FSR) though a catchment characteristic describing the extent of urbanisation was included in the various regression analyses. This report outlines work since publication of the FSR and presents revised procedures to account more satisfactorily for urbanisation. These procedures are particularly relevant to catchments in the 5 to 100 km^2 size range and where urban development is fairly uniformly distributed over the catchment. Outside these conditions, the estimates obtained should be viewed with some caution. In particular, small catchments (<2 km^2) may be better considered using sewer design techniques.

The main recommendations of this report may be summarised as follows.

1.1 The unit hydrograph method

(i) Unit hydrograph time to peak on urban catchments is adequately estimated by the existing equation (FSR Vol 1, p 407).

(ii) Unit hydrograph shape on urban catchments is adequately estimated by the existing triangular shape defined by

$$QpTp = 220.$$

(iii) Percentage runoff on urban catchments is better estimated by

$$PR_u = PR_r \; (1 - 0.3 \; URBAN) + 21.0 \; URBAN$$

where $PR_r = 102.4 \; SOIL + 0.28 \; (CWI - 125) + 0.1 \; (P - 10) - 1.9$

and URBAN is the fraction of the catchment under urban development.

Other variables are as defined in the FSR. The revised soils map given in Supplementary Report No 7 should be used to evaluate SOIL.

(iv) To estimate the T-year flood using the above unit hydrograph and percentage runoff equations, the design input should consist of

CWI	-	the same as given in the FSR (Vol 1, p 465)
Rainfall Duration	-	the same as given in the FSR (Vol 1, p 462)
Rainfall Depth	-	the same as given in the FSR, but with return period equal to that of the required flood.
Rainfall Profile	-	the 50% summer profile.

1.2 The mean annual flood approach

(i) The effect of urbanisation on mean annual flood may be estimated from

$$\frac{\bar{Q}_u}{\bar{Q}_r} = (1 + \text{URBAN})^{1.5} \left(1 + 0.3\ \text{URBAN}\left(\frac{70}{PR_r} - 1\right)\right)$$

where suffices u and r refer to urban and rural conditions respectively

$\bar{Q}_r$ is the prediction from the FSR equation (Vol 1, p 341)

and PR_r is obtained from a simplified form of the equation given under 1.1(iii)

$$PR_r = 102.4\ \text{SOIL} + 0.28\ (\text{CWI}-125)$$

the relevant value for CWI is again found from FSR Vol 1, p 465, figure 6.62. In the Essex, Lee and Thames Region, the recommendations are rather different, and are explained fully in section 5.2 of this report.

(ii) Growth curves of the ratio T-year flood to mean annual flood ($Q_T/\bar{Q}$) against T show some flattening with increasing urbanisation, supporting the intuitive expectation that rarer floods are less affected by urbanisation. Rules for constructing the growth curve for a given region and a given degree of urbanisation are given in this report.

A step-by-step guide and worked example, including a demonstration of how local data may be merged with the above equations, is given in section 6 of this report.

2. THE EFFECTS OF URBANISATION ON FLOOD RESPONSE

The flood potential of a catchment is significantly increased by urbanisation. The introduction of impervious surfaces and an efficient drainage system increases the volume of runoff and reduces flow travel times, yielding a flood hydrograph that is faster to peak, faster to recede and of increased peak discharge. Correspondingly, the flood frequency distribution is affected and floods of all return periods are, in general, increased. The magnitude of the increase, however, depends not only on the extent of urbanisation, but also on the relationship of the urban response to the original rural response. In this respect four factors are of particular significance. Firstly, catchments characterised by low percentage runoff and slow response are more affected by urbanisation than catchments already characterised by high percentage runoff and rapid response. Secondly, urbanisation has a greater effect on the response to small storms which previously yielded low percentage runoff and little overland flow than on the response to severe storms. Consequently, in terms of the flood frequency distribution, mean annual flood will be increased by a greater proportion than rarer floods. Thirdly, because urban catchments respond faster and because they yield runoff from smaller events, the T-year flood after urbanisation tends to be caused by a shorter duration storm, of smaller rainfall depth but higher intensity. Consequently, the effect of urbanisation on T-year flood depends on local rainfall characteristics and, in particular, on the relationship between rainfall intensities for short and long duration storms. Fourthly, the effect of urbanisation depends on the location of urban development within the catchment, which affects both the relative scale and phasing of response from different parts of the catchment : urbanisation in areas which previously contributed little to storm runoff has a greater effect than urbanisation in areas which already used to contribute; urbanisation in upstream areas may result in a rapid urban response which coincides with and reinforces the slower rural response from downstream, while urbanisation in downstream areas may cause the urban response to pass before the slow rural response from upstream arrives.

3. THE FSR TREATMENT OF URBANISATION

Although it was not within the brief of the FSR to investigate the effects of urbanisation, several catchments for which data were available were to some extent urbanised. Therefore URBAN, the fraction of the catchment under urban development, was included as an independent variable in the various regression analyses (URB, the percentage urbanised, i.e. 100.URBAN, and URBT (= 1 + URBAN) were also used, but in this report all equations are written out in terms of URBAN). The aim of the FSR was not so much to allow prediction of the effects of urbanisation, but rather to allow salvage of data that might otherwise have to be rejected. URBAN proved to be a significant variable in the unit hydrograph analysis entering into the recommended equations for both percentage runoff and time to peak. However, in the mean annual flood analysis, URBAN was significant only in the Essex, Lee and Thames region, the only region with an appreciable number of urbanised catchments. The effects of URBAN on the growth curve was not investigated.

In the regression analyses, equations were derived for the various dependent variables in terms of catchment characteristics. These characteristics, although conceptually unrelated, did exhibit some statistical correlation. For example, few urbanised catchments are very steep, and thus some correlation exists between slope and URBAN. Consequently, some of the effect of URBAN may be spuriously accounted for by slope (and vice versa), and the regression coefficients may not accurately estimate the true effect of URBAN and slope alone. Subsequent work has compared the form of the equations derived (i) when URBAN is included and then excluded from the independent variables, and (ii) when separate rural and urban subsets are used. This work has shown that some modifications to both the unit hydrograph and mean annual flood approaches are desirable to account satisfactorily for urbanisation. These modifications are presented below, and a step by step guide to the complete revised procedures (together with details of how to include local data) is given in Section 6. No account is taken in these modificationsof the effect of location of urban development, though this may be considered by: (i) splitting the catchment into subcatchments and using the modified unit hydrograph method presented to obtain subcatchment hydrographs, and (ii) routing the subcatchment hydrographs downstream to the point of interest. Initial work on this approach has been encouraging (Packman, 1978). Work on the effects of urbanisation is continuing under contract for the Department of the Environment.

4. MODIFCATIONS TO UNIT HYDROGRAPH APPROACH

4.1 Time to Peak

Using only the rural catchments from the FSR data set, a new equation for time to peak was derived:

$$Tp_r = 59.5\ S^{-.38}\ RSMD^{-.45}\ L^{.10} \qquad (1)$$

(Throughout this report suffixes r and u will refer to rural and urban conditions respectively.) Full details of the regression are given in the Appendix. Comparing this equation with the FSR equation (Vol 1, p 407):

$$Tp = 46.6\ S^{-.38}\ RSMD^{-.42}\ L^{.14}\ (1 + URBAN)^{-1.99} \qquad (2)$$

it was found that corresponding coefficients are all within one standard error of each other. This suggests that any interaction between URBAN and the other independent variables is within the general noise level. Furthermore, substituting the rural equation (1) into the FSR equation (2) gives:

$$Tp = Tp_r \cdot \{0.783\ RSMD^{.03}\ L^{.04}\ (1 + URBAN)^{-1.99}\}$$

which for typical values of RSMD and L for urban catchments can be reduced to:

$$Tp_u = TP_r\ (1 + URBAN)^{-1.99} \qquad (3)$$

The factor $(1 + URBAN)^{-1.99}$ is thus considered to represent the effect of urbanisation satisfactorily, and no modifications to the existing FSR equation (2) are considered necessary for predicting the effect of urbanisation.

4.2 Unit Hydrograph Shape

Because urban areas within an urbanising catchment respond faster but rural areas continue to respond as before, one might expect a more skewed unit hydrograph with shorter time to peak but the same time base. To test this, new equations were derived for the hydrograph shape functions (QpTp) and W/Tp) based on urban-only and rural-only data sets. Comparing these equations with the original FSR equations (Vol 1, p 401) showed corresponding coefficients were all within one standard error of each other. Consequently the same unit hydrograph shape as given in the FSR is recommended for urban and urbanising catchments, i.e.:

$$QpTp = 220 \quad , \quad TB = 2.525\ Tp \qquad (4)$$

The apparent insensitivity of hydrograph shape to urbanisation does not necessarily mean no change occurs. The effect may exist in small sewered catchments, but become damped in open watercourses downstream. Moreover, any differences in hydrograph shape may be masked by the separation of quick from slow response during analysis.

4.3 Percentage Runoff

A new equation for percentage runoff was derived using only the rural catchments from the FSR data set:

$$PR_r = 102.4\ SOIL + 0.28\ (CWI-125) + 0.10\ (P-10) - 1.9 \qquad (5)$$

Full details of the regression are given in the Appendix. Comparing the rural equation with the FSR equation (Vol 1, p 419):

$$PR = 95.5\ SOIL + 0.22\ (CWI-125) + 0.10\ (P-10) + 12.0\ URBAN \qquad (6)$$

the coefficients of SOIL and CWI differ by about 1.5 and 3 standard errors respectively, suggesting more fundamental differences between urban and rural catchments. Furthermore, use of equations (5) and (6) predicts virtually the same increase in percentage runoff with urbanisation whether the original rural percentage runoff was high or low. Consequently, a new form of percentage runoff equation is proposed for urban areas:

$$PR = PR_r \frac{100-I}{100} + PR_i \frac{I}{100} \qquad (7)$$

where PR_r is the rural percentage runoff

PR_i is the impervious area percentage runoff

and I is the catchment overall percentage imperviousness

Although percentage imperviousness depends on the type of development (city centre, industrial, residential), surveys have shown that for catchments greater than 2 km^2 an average value of I = 30 URBAN (i.e. 100% urbanised $\equiv$ 30% impervious) may be used. With this relationship, the FSR data set gave a mean value for PR_i of 63%. However, preliminary analysis of fully-sewered catchment data suggested a value of 80%, and this was recommended in an earlier paper (Packman, 1977). Subsequent work (Kidd and Lowing, 1979) including more data suggests a value of 70%, and this is the current recommendation. The choice of a value for PR_i may seem somewhat arbitrary, but it is based on as much data as is available. Moreover quite large changes in PR_i generally have only a small effect on the overall percentage runoff estimate.

Substituting for PR_r, PR_i and I, equation (7) becomes:

$$PR = \{102.4\ SOIL + 0.28\ (CWI-125) + 0.10\ (P-10) - 1.9\}\cdot\{1-0.3\ URBAN\} + 70\{0.3\ URBAN\} \qquad (9)$$

This equation applied to the FSR data set yields a standard error of estimate of 15.02, which is a slight improvement over the original FSR equation (6). It predicts increases in PR due to complete urbanisation of + 18% and + 6% respectively for catchments with low (10%) and high (50%) rural percentage runoffs. This equation, however, should not be applied to small, fully-sewered catchments. Floods on such catchments usually arise from short duration summer thunderstorm events, which in general yield very little runoff from pervious surfaces. Such catchments may be better considered using sewer design techniques. Use of equation (9) requires a value of SOIL. The original FSR soils map left some areas unclassified, but a new soils map giving full coverage is presented in Supplementary Report No. 7.

4.4 Design Conditions

To apply the unit hydrograph and percentage runoff equations in design, specifications are required for a combination of antecedent condition and design storm that may be expected to yield a flood peak of the required return period. In the FSR, a simulation technique (Vol 1, ch. 6.7) was used to obtain a consistent set of specifications that would yield flood peaks which matched the complete flood frequency distribution. This required the recommended design storm to have a depth of different return period from the resultant flood peak, a recommendation that has led to some confusion. However, this recommendation was based on mainly rural catchments; urban catchments are generally less variable in response, and thus their flood frequency curves approach the corresponding rainfall frequency curves with increasing urbanisation. This being so, a simpler choice of design conditions for urban catchments should be possible. Based on 11 catchments, comparing the flood frequency curve implied by the particular choice of antecedent condition and design storm with the observed flood frequency curve and, where available, the simulated flood frequency curve of FSR Vol 1, Ch 6.7, the following design conditions were chosen.

CWI - defined from SAAR as per FSR Vol 1, p 465
D - (1 + SAAR/1000) Tp as per FSR Vol 1, p 462
P - as per FSR Vol 1, pp 462-464, but with return period <u>equal</u> to that of required flood
profile - 50% summer.

Compared with the FSR recommendations, the use of equal return periods leads to a flatter flood frequency curve (up to 500 year level at least), and indeed this is borne out by such data as exist (see Section 6.3). Use of the 50% summer rainfall profile results in a slight increase in peak discharge, in most cases less than 5%. The profile is recommended, in part, for consistency with sewer design methods currently in use and under development in the UK.

Following the procedures of Supplementary Report No. 9, the peak of the convolution of the 50% summer profile with the FSR triangular unit hydrograph may be obtained from

$$\hat{q} = RC \cdot \frac{PR}{100} \cdot \frac{P}{D} \cdot AREA \qquad (9)$$

where RC is obtained from the ratio D/Tp - see Figure 1.

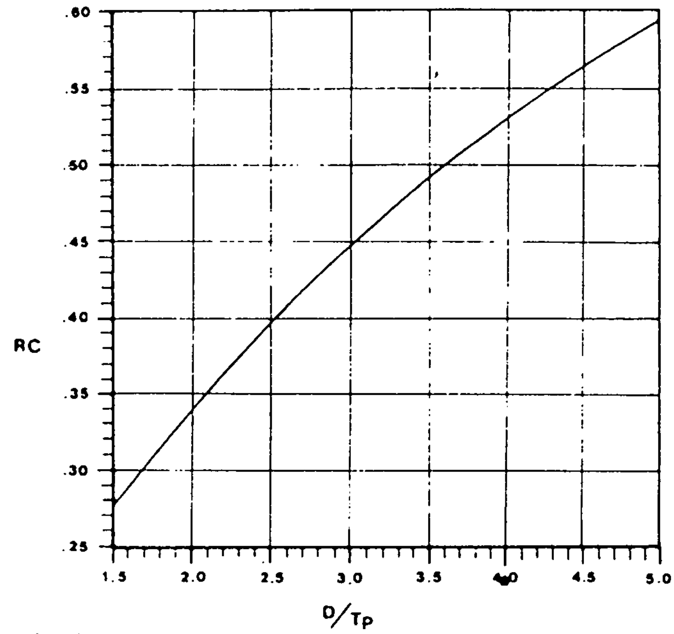

FIGURE 1

Graph of routing coefficient for use in 'Rational' formula - urban catchments

(based on triangular unit hydrograph and 50% summer rainfall profile)

Similarly, the complete hydrograph shape can be obtained from Figure 2, sketching in a hydrograph for the appropriate D/Tp, interpolating at intervals of t/Tp, and multiplying all $q/\hat{q}$ by the $\hat{q}$ value obtained from equation (9).

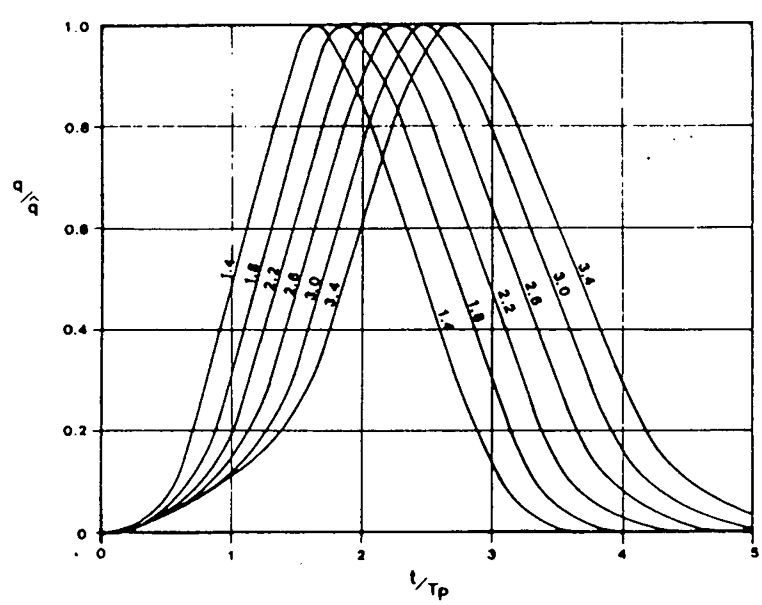

FIGURE 2

Standard hydrograph shapes for stated values of D/Tp urban catchments

5. MODIFICATIONS TO MEAN ANNUAL FLOOD APPROACH

5.1 Mean Annual Flood

URBAN does not appear in the FSR recommended equation for mean annual flood:

$$\bar{Q} = \text{(Regional multiplier) } AREA^{.94} STMFRQ^{.27} SOIL^{1.23} RSMD^{1.03} (1+LAKE)^{-.85} S1085^{0.16} \quad (10)$$

Furthermore, analyses similar to those described under Section 4 for various subsets of the data showed large variation in the effect of URBAN. A compromise equation has been determined, namely:

$$\frac{\bar{Q}_u}{\bar{Q}_r} = (1+URBAN)^{1.8} \quad (11)$$

where $\bar{Q}_r$ is the $\bar{Q}$ estimate obtained from equation (10) above. However, this equation predicts the same effect of urbanisation whatever the character of the original rural response. To overcome this shortcoming, an equation which takes account of the original rural response has been derived from the unit hydrograph method. From equation (9), since RC depends on the ratio D/Tp which is constant for a given catchment.

$$\frac{\hat{q}_u}{\hat{q}_r} = \frac{PR_u}{PR_r} \cdot \frac{P_u}{P_r} \cdot \frac{D_r}{D_u} \quad (12)$$

Assuming the "Average Non-Separated Flow" component is small, and substituting approximate expressions for PR_u/PR_r, P_u/P_r and D_r/D_u gives:

$$\frac{\bar{Q}_u}{\bar{Q}_r} = (1+URBAN)^{2n} \left\{1 + \frac{I}{100} \left(\frac{PR_i}{PR_r} - 1\right)\right\} \quad (13)$$

where n is rainfall continentality (see FSR, Vol 2, p 26)

I is percentage impervious area

PR_i is percentage runoff from impervious area

and PR_r is design percentage runoff from rural area.

Substituting recommended values for I and PR_i (see Section 4.3) and putting $n = .75$ (which is a fair assumption for areas with average annual rainfall in the range 500-1000 mm) gives:

$$\frac{\bar{Q}_u}{\bar{Q}_r} = (1+URBAN)^{1.5} \{1 + 0.3URBAN \ (\frac{70}{PR_r} - 1)\} \qquad (14)$$

This equation is preferred to equation (11) for estimating the effect of urbanisation on mean annual flood. It is presented graphically in Figure 3 for a range of URBAN and PR_r. PR_r can be estimated from equation (5), but since the effect of P on PR_r is small, a good estimate of PR_r may be obtained from the reduced form:

$$PR_r = 102.4 \ SOIL + 0.28\,(CWI-125) \qquad (15)$$

where SOIL is found from the revised soils map (see Supplementary Report No 7)

and CWI is found from SAAR using FSR Vol. 1, Figure 6.62, p 465.

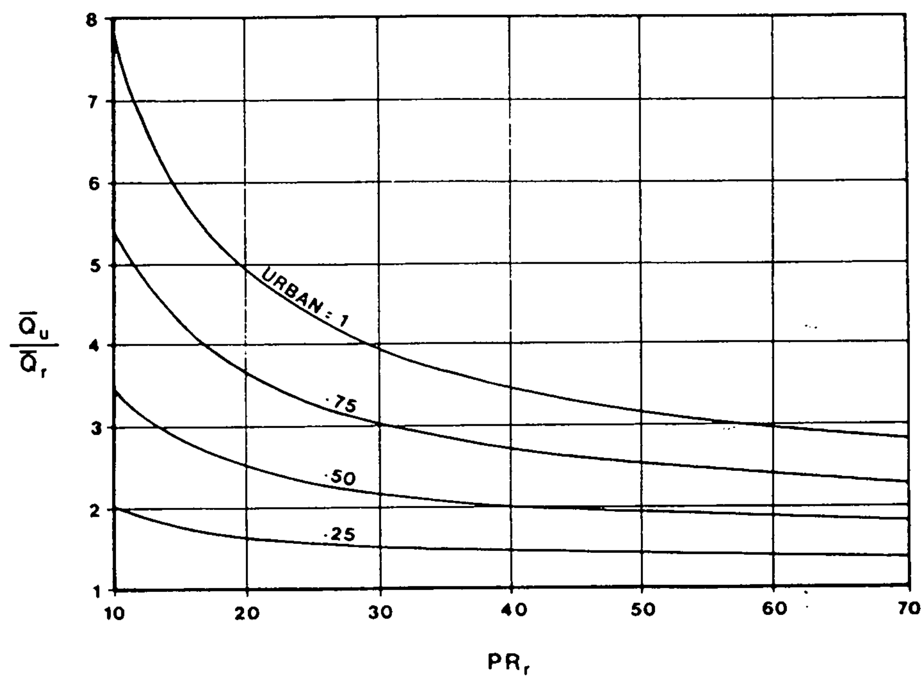

FIGURE 3 Ratio of urban to rural mean annual flood against rural condition percentage runoff

5.2 Mean Annual Flood in the Essex, Lee and Thames region

In the FSR, mean annual flood data from the Essex, Lee and Thames region (region 6) were not well represented by the general equation (10). Consequently a special equation was derived:

$$\bar{Q} = 0.373\ \mathrm{AREA}^{.70}\ \mathrm{STMFRQ}^{.52}\ (1+\mathrm{URBAN})^{2.5} \tag{16}$$

For the reasons discussed in Section 5.1, replacing $(1+\mathrm{URBAN})^{2.5}$ by equations (14) and (15) should yield an improvement in the fit of equation (16) to the data. In fact, a slight worsening of the fit occurs, which could be due to interaction between URBAN and the other independent variables, or to equation (16) yielding a poor estimate of $\bar{Q}_r$. For rural catchments, equation (16) reduces to just two variables, and compared with the general FSR equation, the effects of SOIL, RSMD, LAKE, and S1085 are ignored. Indeed, Supplementary Report No 6 shows that for small (<20 km^2) rural catchments the general equation with the overall national multiplier .0201, is a better fit than equation (16). The best fit multiplier for region 6 rural conditions has subsequently been determined as .0153 (the same as region 5), but using the general FSR equation with this multiplier and adjusting for urbanisation using equations (14) and (15) still yields only the same standard error of estimate as equation (16). Thus, although the general equation seems to fit rural conditions better, the fit of equation (16) to urban conditions has still not been bettered. Indeed, the special form of equation (16) may reflect inconsistencies in mapping such features as STMFRQ between rural and urban catchments. The recommendations for mean annual flood estimation in the Essex, Lee and Thames region can therefore be stated as follows:

(i) For catchments already substantially urbanised, use equation (16).

(ii) For substantially rural catchments to be urbanised, estimate $\bar{Q}_r$ from equation (10) with regional multiplier .0153, and adjust for urbanisation using equations (14) and (15).

5.3 Regional Growth Curve

As already discussed, urbanisation may be expected to flatten the growth curve, an effect not investigated in the FSR. Subsequent examination of the fitted parameter values for the General Extreme Value (GEV) distribution has indeed shown a tendency to flattening of the growth curve with increasing urbanisation.

However, GEV parameters are subject to large uncertainties, so catchments were pooled into three degrees of urbanisation (URBAN = .25, .5, .75) and national growth curves derived. These curves were subsequently smoothed and are presented in Figure 4 along with the FSR national growth curve (Vol. 1, p. 181), which is considered to represent URBAN = 0. The urban curves are based on quite short lengths of total record (44 years for URBAN = .75, 70 years for URBAN = .50, 300 years for URBAN = .25), and consequently have only been plotted to a return period of 50 years. Figure 4 indicates the general effect of urbanisation on growth curves but use of these urban curves with individual region curves (to represent rural conditions) is not recommended; anomalies can arise in some regions with the urban growth curves obtained being steeper than the original rural growth curves.

Any attempt to overcome the anomalies must to some extent be intuitive because of the lack of data. The approach adopted has been to consider an equivalent return period such that the growth factor for the T year flood on an urban catchment is found at an equivalent return period T' years on the rural growth curve. The equivalent return periods are presented in Table 1 as equivalent y-values, where y (the Gumbel reduced variate) is related to return period, T, by:

$$y = -\ln\left(-\ln\left(1 - \frac{1}{T}\right)\right) \tag{17}$$

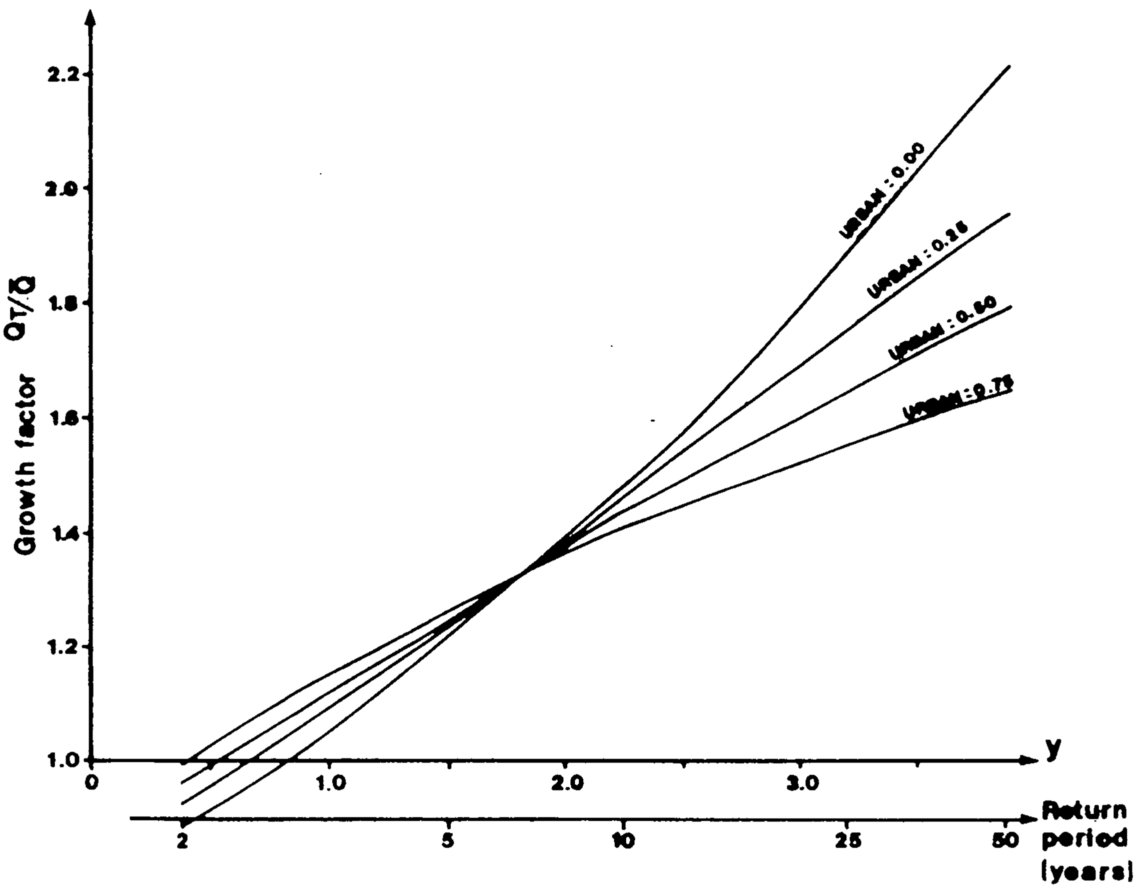

FIGURE 4 The effect of URBAN on growth rate

TABLE 1. Equivalent y-values for specified return periods and values or URBAN

URBAN	RETURN PERIOD, T					
	2	5	10	20	25	50
.00	.37	1.50	2.25	2.97	3.20	3.90
.25	.52	1.55	2.20	2.76	2.93	3.35
.50	.65	1.60	2.12	2.55	2.67	3.00
.75	.78	1.65	2.04	2.35	2.43	2.67

The equivalent y-values may be used with FSR Vol 1, Fig 2.14. p 174 to determine growth factors. However, it may be easier to interpolate in Table 2.

TABLE 2. Regional growth factors (g.f.) at intervals of y

REGION	y								
	0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
1	.82	.94	1.06	1.20	1.36	1.53	1.72	1.94	2.17
2	.84	.94	1.05	1.18	1.33	1.51	1.72	1.95	2.23
3	.84	.98	1.11	1.25	1.38	1.52	1.65	1.79	1.92
4	.80	.93	1.07	1.23	1.40	1.58	1.79	2.01	2.25
5	.79	.93	1.10	1.29	1.52	1.79	2.11	2.49	2.93
6/7	.77	.92	1.09	1.28	1.50	1.74	2.02	2.34	2.69
8	.78	.92	1.07	1.23	1.40	1.58	1.76	1.95	2.16
9	.84	.96	1.08	1.21	1.35	1.49	1.64	1.80	1.97
10	.85	.96	1.07	1.19	1.31	1.45	1.58	1.73	1.88

The equivalent return period concept, like the growth curves of Figure 4, has been taken to a return period of 50 years. Extension of the urban growth curves beyond this point is highly subjective since virtually no data exist. However, it is generally considered that the effect of urbanisation is reduced with increasing return period, and that as T becomes larger the T-year flood after urbanisation leads to the same value as the T-year flood before urbanisation. One way to achieve such an effect is to fit an exponential decay to the ratio of urban to rural T year flood. After several trials, the following form was chosen:

$$Q_{Tu}/Q_{Tr} = 1 + Be^{-ky} \qquad (18)$$

where Q_T is the T-year flood

y is the Gumbel reduced variate - see equation (17)

and B & k are constants

This equation was fitted to the ratio Q_{Tu}/Q_{Tr} at T = 6.6 years (chosen since at this point Q_{Tu}/Q_{Tr} = 1) and T = 50 years.
The corresponding expressions for k and B are

$$k = .48 \left\{ \ln \left(\frac{\bar{Q}_u}{\bar{Q}_r} - 1 \right) - \ln \left(\frac{Q_{50u}}{Q_{50r}} - 1 \right) \right\} \qquad (19)$$

$$B = \left(\frac{Q_{50u}}{Q_{50r}} - 1 \right) e^{3.9k} \qquad (20)$$

Then, since $y \approx \ln T$ for large T, equation (18) may be rewritten

$$Q_{Tu}/Q_{Tr} = 1 + BT^{-k} \tag{21}$$

These equations (19), (20) and (21) may be used to extend the growth curve beyond 50 years, though it must be stressed that the procedure is largely intuitive.

6. STEP BY STEP GUIDE TO THE REVISED PROCEDURES

In the following sections a worked example of the revised procedures for urban catchments is presented. The catchment used is the Almond at Almond weir (Hydrometric No. 19002) a 43.8 km^2 catchment to the west of Edinburgh. Using both the unit hydrograph and the flood frequency approaches an estimate is made of the 10-year flood and the 100-year flood assuming the urban content of the catchment is to increase from 14% to 60%. All catchment characteristics and local data have been taken from the FSR Vol 4.

6.1 The Unit hydrograph approach

(i) Determine catchment characteristics AREA, L, S, RSMD, URBAN as per Vol. 1, p. 458-460, steps 1 to 5.

AREA = 43.8 km^2
L = 17.89 km
S = 5.06 m/km
RSMD = 41.3 mm
URBAN = 0.14 (Now)
= 0.60 (Future)

(ii) Determine the time to peak of the 1 hour unit hydrograph

$$Tp = 46.6\, L^{0.14} S^{-0.38} RSMD^{-0.42} (1+URBAN)^{-1.99}$$

Tp (future) = 3.10 hr

If local data have enabled a better estimate of Tp, this may be adjusted for future urbanisation by

From FSR Vol 4, p. 35
Excluding event 7
Mean Tp observed = 7.03 hr

$$Tp(future) = Tp(now) \cdot \left\{\frac{1+URBAN(future)}{1+URBAN(now)}\right\}^{-1.99}$$

Tp (future) = 3.58 hr

(iii) Determine data interval, T, such that

$T \leq Tp/5$

T = 0.5 hr

(iv) Adjust Tp for the one hour unit hydrograph to Tp' for the T hour unit hydrograph, ie

$$Tp' = Tp + (T-1)/2$$

Tp' = 3.33 hr

(v)	Determine design rainfall duration, D, such that	SAAR = 1062 mm
	$D = (1 + SAAR/1000)Tp$	D = 6.87 hr
	and round such that D is nearest odd integer multiple of T.	D = 6.5 hr
(vi)	Choose storm return period equal to required flood return period	(i) T = 10 years (ii) T = 100 years
(vii)	Determine design rainfall depth P of duration D and return period T as per FSR Vol 1, p 462-464, step 11.	M52D = 65.9 mm r = 0.25 M5(6.5H)/M52D = 0.51 M5(6.5H) = 33.6 mm 10 year growth factor = 1.165 100 year growth factor = 1.82 ARF = 0.92 P(10 years) = 36.0 mm P(100 years) = 56.3 mm
(viii)	Determine catchment wetness index, CWI, and soil index, SOIL, as per FSR Vol. 1, p. 465, steps 12 and 13	CWI = 124 SOIL = 0.45
(ix)	Determine design percentage runoff	
	$PR_r = 102.4\ SOIL + .28(CWI - 125) + .10\ (P-10) - 1.9$	PR_r (10 years) = 46.5
	$PR_u = PR_r\ (1 - .3URBAN) + 70(0.3URBAN)$	PR_u (10 years) = 50.7
	If local data for several significant events (relative to either rural or urban conditions) are available, a better estimate of PR_u may be obtained as follows.	
	(a) Correct data back to rural condition, if necessary, using $PR' = \frac{PR_{OBS} - 70\ \{0.3\ URBAN\}}{1 - 0.3\ URBAN}$	From FSR Vol 4, p 35. URBAN = 0.14 excluding event 7 PR' = 53.0, 49.2, 45.5, 66.9, 56.0, 75.4, 56.4, 61.6, 47.5, 59.6
	This assumes I/100 = 0.3 URBAN, a local estimate of I/100 may be used in place of 0.3 URBAN	No local estimate available
	(b) Correct PR' values to standard conditions (CWI = 125, P = 10), and find mean standard percentage runoff SPR	SPR = Mean {53.8, 46.8, 43.6, 67.0, 53.8, 74.9, 56.1, 59.2, 62.1, 57.7}
	$SPR = Mean\ \{PR' - 0.28(CWI_{obs} - 125) - 0.10(P_{obs} - 10)$	SPR = 57.5

(c) Substitute SPR for 102.4SOIL - 1.9 in equation for PR_r	PR_r (10 years) = 59.8 PR_r (100 years) = 61.9
(d) Adjust to PR_u as above using local estimate for I/100 if available	No local estimate for I/100 PR_u (10 years) = 61.6 PR_u (100 years) = 63.4
(x) Determine average non-separated flow, ANSF, from $\text{ANSF} = \{0.00033(\text{CWI}-125) + 0.00074\ \text{RSMD} + 0.003\}\cdot\text{AREA}$	ANSF = 1.5 m^3/s
(xi) If a peak flow estimate only is required, this may be obtained from $\hat{q} = \text{RC}\cdot\frac{\text{PR}}{100}\cdot\frac{\text{P}}{\text{D}}\cdot\text{AREA}$ $\hat{Q} = \hat{q} + \text{ANSF}$ where RC is found from the ratio D/Tp' - see Figure 1 Fig 1 assumes QpTp = 220. if local data suggest a different unit hydrograph shape steps (xiii) to (xvii) should be followed.	D/Tp' = 1.95 RC = 0.335 $\hat{q}$ (10 years) = 50.1 m^3/s $\hat{q}$ (100 years) = 80.6 m^3/s $\hat{Q}$ (10 years) = 51.6 m^3/s $\hat{Q}$ (100 years) = 82.1 m^3/s
(xii) If a complete design hydrograph is required, this may be found from fig 2, sketching in a curve for the respective D/Tp', interpolating at intervals of T/Tp', multiplying all flow ordinates by $\hat{q}$ obtained above, and adding ANSF. Fig 2 also assumes QpTp = 220. If local data suggest a different unit hydrograph shape, steps (xiii) to (xvii) should be followed.	D/Tp' = 1.95 T/Tp' = 0.15 $q/\hat{q}$ = {.005, .010, .035, .065, .110, .180, .295, .430, .570, .710, .840, .950,...} q(10 years) = {.3, .5, 1.7, 3.3 5.5, 9.0, 14.8, 21.5, 28.6, 35.6,42.1,47.6..} Q(10 years) = {1.8, 2.0, 3.2, 4.8...}
(xiii) Where local data have allowed an estimate of unit hydrograph shape this may be adjusted for urbanisation by multiplying all flow and dividing all time by the ratio Tp(now)/Tp(future)	From FSR Vol 4, p 35 (excluding event 7) mean Qp = 18.1 Converting for use with percentage runoff (FSR Vol 1, p 422) gives Qp = 22.1 For triangular unit hydrograph TB = 555.6/Qp = 25.1 From step (iv) Tp(now)/TP(future)=2.11 Thus Qp(future) = 46.6 TB (future) = 11.9
(xiv) Interpolate unit hydrograph ordinates at interval T, as FSR Vol 1, p 468, step 17.	T = 0.5, unit hydrograph ordinates are {7.0, 14.0, 21.0, 28.0, 35.1, 42,0, 45.7, 43.0, 40.2 ...}

(xv) Distribute rainfall depth P across duration D according to the 50% summer profile reproduced in Figure 5 as cumulative percentage of depth against percentage of duration from the beginning of the storm (unlike the FSR where the profile is reproduced as percentage of depth in central percentage of storm duration).

Since D = 6.5
and T = 0.5
% of D per T = 7.7
Rainfall profile found for 10 year case only

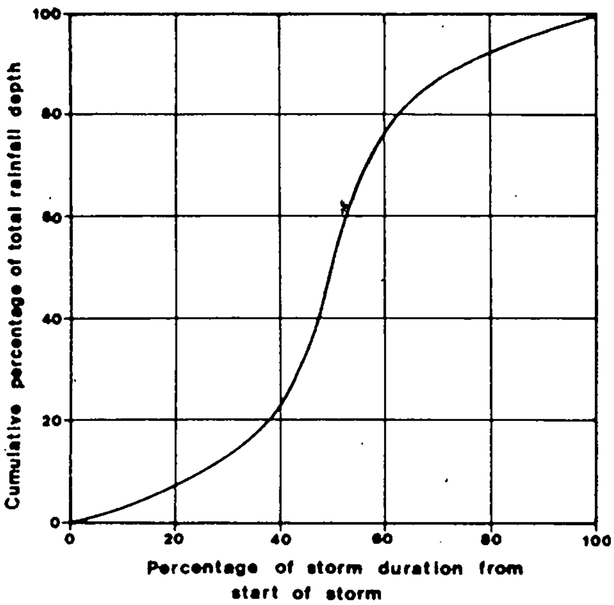

FIGURE 5

50% summer cumulative rainfall profile

% of Duration	7.7	15.4	23.1	30.8	38.5	46.2	53.9	....
Cumulative % of P	2.5	5.5	9.0	14.0	21.5	37.0	63.0	....
Incremental % of P	2.5	3.0	3.5	5.0	7.5	15.5	26.0	....
mm depth (10 years)	0.9	1.1	1.3	1.8	2.7	5.6	9.4	....

(xvi) Obtain net rainfall profile by multiplying by PR_u

{0.6, 0.7, 0.8, 1.1, 1.7, 3.4, 5.8, 3.4, ...}

(xvii) Convolute net rainfall with unit hydrograph and add ANSF as per FSR Vol 1, p 468-9, steps 18 and 19

$\hat{q}$ (10 years) = 37.5 m^3/s

Q (10 years) = 39.0 m^3/s

$\hat{Q}$ (100 years) = 61.9 m^3/s

6.2 Mean annual flood approach

(i) Determine catchment characteristics AREA, STMFRQ, SOIL, RSMD, LAKE, S1085, URBAN (see FSR, Vol 1, pp 316-317 and Table 4.19, p 336)

AREA = 43.8 km^2
STMFRQ = 1.07
SOIL = 0.45
RSMD = 41.3 mm
LAKE = 0.0
S1085 = 5.06 m/km
URBAN = 0.14 (now)
= 0.60 (future)

(ii)	Determine regional multiplier (see FSR Vol 1, p 341). For catchment in Essex, Lee and Thames region refer to section 5.2 of this report.	0.0213
(iii)	Determine rural mean annual flood from :	
	$\bar{Q}_r$ = (regional multiplier)$AREA^{.94}$ $STMFRQ^{.27}SOIL^{1.23}$ $RSMD^{1.03}$ $(1+LAKE)^{-.85}S1085^{.16}$	17.0 m^3/s
	If local data are available, relating to rural or urban conditions, a better estimate of $\bar{Q}_u$ may be made as follows.	
	(a) adjust data back to rural conditions, if necessary, using $$\bar{Q}_r = \frac{\bar{Q}_u}{(1+URBAN)^{1.5}\{1+0.3URBAN(\frac{70}{PR_r}-1)\}}$$	From FSR Vol 4, p 136 BESMAF (ie $\bar{Q}_u$) = 15.02 m^3/s URBAN = 0.14
	where: PR_r = 102.4SOIL + .28CWI - 125	PR_r = 45.8
	A local relationship for I may be used in place of .3URBAN.	No relationship for I/100
	And local data for PR_r may be included as per section 6.1 Step (ix)	Following section 6.1, step (ix) PR_r (2.33 years) = 58.9 $\bar{Q}_r$ = 12.2 m^3/s
	(b) Correct for location. Providing the gauging station is in the same catchment as the design location, correct $\bar{Q}_r$ by the ratio of the two catchment areas.	No correction
(iv)	Adjust $\bar{Q}_r$ for the effects of urbanisation by $$\bar{Q}_u/\bar{Q}_r = (1+URBAN)^{1.5}\{1+0.3URBAN(\frac{70}{PR}-1)\}$$	SAAR = 1062 mm CWI = 124 PR_r = 58.9 - see (iiia) above
		$\bar{Q}_u/\bar{Q}_r$ = 2.09
	where PR_r = 102.4 SOIL + 0.28(CWI-125), (or from local data following approach of Section 6.1 step (ix))	$\bar{Q}_u$ = 25.5 m^3/s
	and CWI is defined from SAAR, see FSR, Vol 1, Figure 6.62, p.465.	

(v) For T-year flood (where $T \leq 50$) interpolate in Table 1 of this report to find equivalent y-value for chosen return period and value of URBAN. Enter Table 2 for specific region and equivalent y-value to find growth factor (gf). T-year flood is then given by

$Q_{Tu} = (gf) \cdot \bar{Q}_u$

$y = 2.09$
Region 2
$gf = 1.36$

$Q_{10u} = 34.7\ m^3/s$

(vi) For T-year flood (where T is greater than 50). Follow step (v) above for T = 50, to determine Q_{50u}. Repeat for URBAN = 0 to determine Q_{50r}. Form ratio Q_{50u}/Q_{50r}.

URBAN = 0.60, $y = 2.87$
$gf = 1.66$, $Q_{50u} = 42.3\ m^3/s$
URBAN = 0, $y = 3.9$
$gf = 2.17$, $Q_{50r} = 26.5\ m^3/s$

$Q_{50u}/Q_{50r} = 1.60$

(vii) Use $\bar{Q}_r$ from step (iii) and regional growth curves of FSR Vol 1, p 174, to determine Q_{Tr}.

$Q_{100r} = 2.63 \cdot \bar{Q}_r = 32.1\ m^3/s$

(viii) Knowing $\bar{Q}_u/\bar{Q}_r$ from step (iv) and Q_{50u}/Q_{50r} from step (vi), estimate Q_{Tu} from $Q_{Tu} = Q_{Tr}(1 + BT^{-k})$

where

$$k = 0.48\{\log_e(\frac{\bar{Q}_u}{\bar{Q}_r} - 1) - \log(\frac{Q_{50u}}{Q_{50r}} - 1)\}$$

and $$B = (\frac{Q_{50u}}{Q_{50r}} - 1)\, e^{3.9k}$$

$k = 0.48\{0.862 + 0.5108\} = 0.287$

$B = 1.84$

$Q_{100u} = 1.49\ Q_{100r} = 47.9\ m^3/s$

6.3 Comparison of Estimates

In summary of the above example, the estimates of Q_{10} and Q_{100} obtained in step (xvii) of the unit hydrograph method were 39.0 m^3/s and 61.9 m^3/s respectively. The corresponding estimates obtained from steps (v) and (viii) of the mean annual flood approach were 34.7 m^3/s and 47.9 m^3/s. Agreement between the two methods in this example is quite good. As a general rule, both methods may be expected to yield similar percentage increases in mean annual flood due to urbanisation, but the unit hydrograph method may be expected to yield more reliable estimates of the growth factor $Q_{Tu}/\bar{Q}_u$.

Inclusion of local data to try and improve the estimates is always recommended. However, it is interesting to note, in this example, the effect of such information. If no local data had been included, agreement between the two methods would still have been quite good, but both would, in this case, have given larger estimates. The unit hydrograph method would have given estimates of 46.0 m^3/s and 79.2 m^3/s while the mean annual flood method would have given estimates of 51.1 m^3/s and 70.6 m^3/s. In the unit hydrograph method, local data showed that the catchment response was not quite as fast as predicted, but that volume of runoff was much greater than predicted. Use of predicted percentage runoff would have underestimated both volume and peak discharge by about 30%. However, local information on unit hydrograph shape showed the predicted shape (given by QpTp = 220) led to over-estimates in peak discharge also of about 30%. Thus, in terms of peak discharge, the two errors would have virtually cancelled each other out. Unit hydrograph shape is not normally as important as in this example; in only about 5% of catchments will errors of this magnitude occur. In the mean annual flood method eight years of local data are used to determine a $\bar{Q}_r$ estimate of 12.2 m^3/s. This estimate is much to be preferred to the estimate of 17.0 m^3/s given by the equation. The expected accuracy of the $\bar{Q}$ equation is discussed in FSR Vol 1, p 342.

7. CONCLUSION

The above procedures form the recommendations for design flood estimation in urban and urbanising catchments. Research into the effects of urbanisation is continuing, particularly for smaller (2 to 10 km^2) catchments and for the effect of location of urban development. Research is also underway on the use of balancing reservoirs to mitigate the effects of urbanisation.

8. REFERENCES

Kidd, C.H.R. and Lowing, M.J., 1979. The Wallingford urban subcatchment model, *Institute of Hydrology Report No 60.*

Packman, J.C., 1977. The effect of urbanisation on flood discharges - Discussion and recommended procedures, *MAFF Conf. of River Engineers*, Cranfield.

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9. APPENDIX

Regression details for time to peak and percentage runoff, rural catchments only.

Variable Name	Coeff	Standard Error	t Statis.	R^2	Standard Error of Est.	Constant	Antilog of Const.
TIME TO PEAK OF ONE-HOUR UNIT HYDROGRAPH							
NO OF OBSERVATIONS - 106							
LOG(S1085	-0.38	0.07	5.23	0.78	0.15	1.77	59.5
LOG(RSMD)	-0.45	0.13	3.52	-	-	-	-
LOG(MSL)	0.10	0.05	1.84	-	-	-	-
PERCENTAGE RUNOFF							
NO OF OBSERVATIONS - 1074							
SOIL	102.37	5.82	17.6	[illegible]	15.4	-1.9	
(CWI-125)	.28	.02	12.8	-	-	-	
(P-10)	.10	.02	5.3	-	-	-	

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Flood prediction for small catchments

1. INTRODUCTION

Section I.4.3.7 of the FSR described some simple tests which were used to investigate whether the mean annual flood on small and large catchments could be predicted by the same equation. This was done by splitting the data into groups of large and small catchments to see if this significantly improved the fit. The conclusions were:

(i) that the overall improvement in prediction was small

and that

(ii) floods on small catchments were less well predicted than on large ones.

In the period since the publication of the report, it has become clear that small catchments form the majority of cases in which the FSR methods are being used and that there is some doubt among users as to their applicability in these situations. (Small has been taken to mean catchments with areas of less than 20 km^2). This supplementary report is the result of more detailed investigations of the data.

2. THE AVAILABLE DATA

Fifty-three catchments have been used in regressions of the mean annual flood on catchment characteristics and twenty-three for unit hydrograph analysis. There are, of course, many more stations which have records but which do not have useful ratings at flood levels. The distribution of the fifty-three catchments by size is given below:

Area (km^2)	Number of catchments
< 20	53
< 15	42
< 10	25
< 5	12
< 1	4

Other points to note about the data are that only eleven of the catchments have slopes (S1085) of less than 10m/km and only twelve have SOIL indices of less than 0.45 (the maximum possible SOIL index is 0.5). As the number of catchments decreases in the smaller area bands, they also become less representative; for instance, three of the four catchments of less than 1 km^2 are of only a few hectares high up in the northern Pennines. Of the twenty-three catchments used in the unit hydrograph analysis only four have SOIL indices of less than 0.45, two of these being heavily urbanised and only three have S1085 values of less than 10m/km.

It is clear from these comments that even the fifty-three catchments form a relatively limited set of data on which to base regression estimates of the mean annual flood, the time to peak of the unit hydrograph and percentage runoffs.

Further problems arise in the extraction of catchment characteristics from maps. The first concerns the subjective nature of the mapping of rivers on Ordnance Survey maps which lead to difficulties in the measurement of mainstream length and number of stream junctions towards the source of rivers. This does not matter greatly if the catchment is large but may radically alter the measured characteristics on a small catchment if, for instance, it is a choice between two stream junctions or three. The other main problem is the low resolution of the winter rain acceptance potential (SOIL) map which may overlook large local variations in soil type which are important if they coincide with a small catchment under study. With all these problems in mind, the next two sections deal with the results of analyses of the available data.

3. MEAN ANNUAL FLOOD AND GROWTH CURVES

Six of the catchments in the data set had more than 10 percent urban development; these were not used in the analysis. To see if the equations recommended in Section I.4.3.10 of the FSR for calculation of the mean annual flood were any less successful for small catchments than for larger ones, the mean annual flood was calculated by this method for the forty-seven non-urban small catchments. The differences between the observed and predicted $\bar{Q}$ were used to calculate a 'standard error' analagous to that obtained from regressions; it was 0.239 compared to the 0.168 quoted for all catchments in the FSR. However, much of the error was attributable to the six catchments in the Thames, Lee and Essex region so the error was recomputed using the national 6-variable equation with the average national multiplier for these six stations. The resulting standard error was 0.183, not much greater than the error on all catchments. All six of the catchments in the Thames, Lee and Essex region were better predicted by this method.

The results of regressing the mean annual flood on catchment characteristics for the forty-seven small non-urban catchments are shown below:

3-variable equation:

$$\bar{Q} = 0.00066\ \mathrm{AREA}^{0.92}\ \mathrm{SAAR}^{1.22}\ \mathrm{SOIL}^{2.0} \qquad (1)$$

$$R^2 = 0.93$$

Standard Error = 0.198
Factorial Error = 1.58

4-variable equation:

$$\bar{Q} = 0.0288\ \mathrm{AREA}^{0.90}\ \mathrm{RSMD}^{1.23}\ \mathrm{SOIL}^{1.77}\ \mathrm{STMFRQ}^{0.23} \qquad (2)$$

$$R^2 = 0.94$$

Standard Error = 0.185
Factorial Error = 1.53

This compares with the already mentioned regional 6-variable equation from the FSR which (except for the Thames, Lee and Essex) is:

$$\bar{Q} = \mathrm{const}\ \mathrm{AREA}^{0.94}\ \mathrm{STMFRQ}^{0.27}\ \mathrm{SOIL}^{1.23}\mathrm{RSMD}^{1.03}\ (1+\mathrm{LAKE})^{-0.85}\mathrm{S1085}^{0.16}$$

$$R^2 = 0.91$$

Standard Error = 0.168
Factorial Error = 1.47

In terms of standard error the 4-variable small catchment equation is rather less precise than the 6-variable regional equation in the FSR. For the small catchments no further variables significantly improved the regressions.

A further comparison was made by splitting the forty-seven catchments into the twelve with SOIL values of less than 0.45 and the thirty-five with SOIL values greater. The best regressions on the two sets produced standard errors of 0.143 for the catchments with the high SOIL values and 0.256 for those with low SOIL values. The standard errors from the FSR regional equations (with Thames, Lee and Essex catchment treated as before) applied to these two groups are 0.144 and 0.275.

The conclusions to be drawn from these regressions are:

(a) There is little difference with small catchments between using the regional equations recommended in the FSR and the 4-variable equation derived directly from the small catchment data.

(b) Small catchments with SOIL indices greater than 0.45 (ie, on soil types 4 and 5) are well predicted by both methods while those on soil types 1, 2 and 3 are not.

(c) Small catchments in the Thames, Lee and Essex region are better predicted by the 6-variable FSR equation with the average national multiplier than by the 3-variable Thames, Lee and Essex equation.

Growth curves

There was insufficient data from small catchments to form regional curves of $Q/\bar{Q}$ so one pooled curve was calculated for all small catchments and compared with the Great Britain curve of section I.2.6.3; Figure 1 shows the comparison. There is a close agreement between the two curves and it seems reasonable to infer from this that not only does the small catchment curve follow the Great Britain curve but that, if the data were available, regional small catchment curves would follow the FSR regional curves.

4. UNIT HYDROGRAPH ANALYSIS

Three hundred and four storm events on the twenty-three catchments were analysed to provide estimates of the time to peak (Tp) of the one hour unit hydrograph and the percentage of the rainfall which appeared as quick response runoff (PR). The regression of percentage runoff on catchment and storm characteristics gave:

$$PR = 76.7SOIL + 5.16URBAN + 0.2(CWI-125) + 0.04(P-10) + 12$$

$$R^2 = 0.386$$

Standard Error = 16.8%

compared to that from 1447 events on one hundred and thirty-eight catchments of all sizes in the FSR:

$$PR = 95.5SOIL + 12URBAN + 0.22(CWI-125) + 0.1(P-10)$$

$$R^2 = 0.432$$

Standard Error = 15%

There is little difference in the standard error of the two equations although the coefficients of the variables differ. Assuming an average SOIL value of about 0.4, it is apparent that the decrease in the coefficient of SOIL is balanced by the appearance of the constant +12. This is probably the result of the small number of low SOIL type catchments available. The standard error of the FSR equation applied to the small catchment data is 16.9%

For Tp the results were, for the small catchment data:

$$Tp = 9.8\ S1085^{-0.32}\ MSL^{0.19}\ (1+URBAN)^{-1.97}\ RSMD^{-0.07}$$

$$R^2 = 0.578$$

Standard Error = 0.16
Factorial Error = 1.45

and for the full data set

$$Tp = 46.6\ S1085^{-0.38}\ MSL^{0.14}\ (1+URBAN)^{-1.99}\ RSMD^{-0.4}$$

$$R^2 = 0.780$$

Standard Error = 0.15
Factorial Error = 1.41

Again the errors are very similar; this time the change in constant is compensated by the much smaller coefficient for RSMD. The standard error of the FSR equation applied to the small catchment data is 0.16.

To give an idea of the accuracy of the unit hydrograph design method applied to small catchments, the mean annual flood on twenty-seven catchments was compared to that predicted by the design method outlined in Section I.6.8.2. The 'standard error' was 0.17 - a factorial error of 1.49. Prediction of ten-year floods on only eight stations yielded a 'standard error' of 0.175 (a factorial error of 1.50).

5. CONCLUSIONS AND RECOMMENDATIONS

Although it seemed initially likely that improvements could be made in the prediction of floods on small catchments by adopting separate equations more detailed analysis has refuted this. With small data sets as used here it is important not to overlook the possibility of large proportions of the error being due to a few stations only, as was found in the case of the stations in the Thames, Lee and Essex region. When account is taken of this, the FSR regional equations are seen to predict as well as equations derived from the small catchments alone. This applies to mean annual flood, time to peak and percentage runoff equations. With the greater confidence in the coefficients given by the FSR equations derived from a much wider data set, it would be unwise to abandon these in favour of equations, which at best, perform only marginally better on the small catchment data sets. Accordingly the following recommendations are made:

(1) The regional equations for mean annual flood should be used (for small catchments) as given in the FSR. The FSR equations for Tp and PR should also be used as before.

(2) In the Thames, Lee and Essex region, it is better to use the FSR mean annual flood equation with the average national multiplier than to use the Thames, Lee and Essex equation as given in the FSR.

(3) Note should be taken of the differences in standard error involved in

estimating the mean annual flood for catchments with SOIL indices of greater and less than 0.45

(4) The regional growth curves of Section I.2.6.3 apply to small catchments equally as to larger ones.

(5) The 3 or 4-variable equations given as equations (1) and (2) could be used if time were at a premium for small catchments, provided that they do not have characteristics outside the range used here.

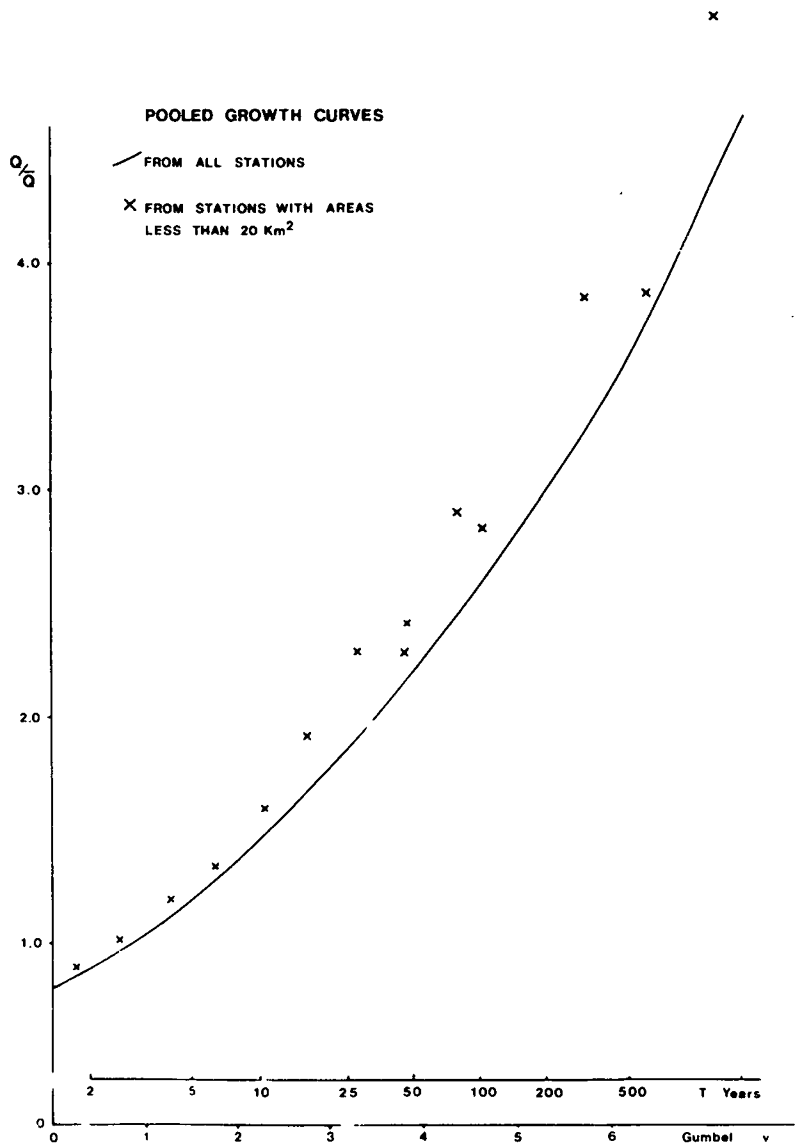


FIGURE 1

Flood Studies Supplementary Report No 7 April 1978

A revised version of the Winter Rain Acceptance Potential (SOIL) Map

The maps of Winter Rain Acceptance Potential provided as Figures I.4.18(S), I.4.18(N) and I.4.18(I) in Vol. V of the FSR have been revised as detailed below. Copies of the revised maps are included with this report.

The majority of the revisions are on Figure I.4.18(S) and that part of Figure I.4.18(N) covered by the Soil Survey of England and Wales. They have been made following discussions between the staff of the Institute and the Soil Survey in the light of new soils data and feedback of hydrological information. Much of the background to this is given in Special Survey No. 11 of the Soil Survey[1] (incorrectly referred to as Special Survey No. 6 in the list of papers contained in the first batch of Supplementary Reports).

The other revision to the maps is that the previously unclassified major urban areas have now been classified. This classification relies quite heavily on correlation between geology and soil type as soils information in urban areas is limited. No additional surveying has been done for this purpose. This mapping of the urban areas has been carried out as part of the input to the urban drainage design package being developed by the Hydraulics Research Station and the Institute of Hydrology.

[1] Farquharson, F.A.K., Mackney, D., Newson, M.D. and Thommasson, A.J. "Estimation of runoff potential of river catchments from soil surveys" Special Survey No. 11. Soil Survey of England & Wales, Harpenden, 1978

Flood Studies Supplementary Report No 8 May 1978

A comparison between the Rational Formula and the Flood Studies Unit Hydrograph procedure

1. INTRODUCTION

The rational or Lloyd-Davies formula, equation (1), remains in use, especially for small catchment work, despite recent advances in flood estimation techniques.

$$Q = C\,i\,A \tag{1}$$

where Q is the flood peak in cusecs
C is a runoff coefficient
i is design rainfall intensity in inches/hour
A is area in acres.

The coincidental dimensional accuracy of the formula is well known but less well known is the fact that the formula can be regarded as the outcome of applying a rectangular unit hydrograph to a uniform rainfall. This suggests that a comparison can be made between peak discharges obtained from this method and those obtained from the unit-hydrograph-based procedure described in Chapter I.6 of the FSR.

The general conclusions of this comparison are that, subject to an assumed use of identical runoff coefficients for small lowland catchments the rational formula will yield flood peaks typically twice as large as those from the FSR. The two methods tend to a greater similarity for larger and steeper catchments. The major source of difference is attributable to the use of the Bransby-Williams formula for design rainstorm duration.

The following sections elaborate the comparison under each source of difference: flood formula (incorporating unit hydrographs and rainstorm profile), runoff coefficient, rainfall return period, rain storm duration and rain storm depth. The justification for the particular choice of design variables for the FSR procedure are fully detailed in Section I.6.7 and will not be repeated here beyond stating that they have been checked against a considerable quantity of UK recorded flow data. As far as is known, there has been no similar attempt in the United Kingdom to calibrate (or check) the rational formula.

2. FLOOD FORMULA

Equation (1) can be rewritten in metric units, with the subscript R denoting use in the rational formula, as

$$Q_R = 0.28 C_R\, i_R\, A \tag{2}$$

where Q_R is in cumecs
C_R is the runoff coefficient
i_R is in mm/hr
A is in km^2

The formula that would be obtained by convoluting a 1 hour rectangular unit hydrograph with a continuing rainfall intensity of i mm/hr over A km^2 is:

$$Q = 0.278\ C\ i\ A \tag{3}$$

thus confirming the closeness of the dimensional approximation in Equation (1).

It is possible to express the 'curve number' approach to unit hydrograph convolution in similar terms to Equation (2). Figure 6.64 of the FSR allows for the effect of the triangular unit hydrograph and profile shape and yields the peak discharge of the convoluted hydrograph Q_F in cumecs/100km^2/10mm of input rainfall with the subscript F denoting the FSR method

$$Q_F = \frac{CN\ C_F\ A\ P_F}{10^3} \tag{4}$$

where $CN/10^3$ is the peak discharge in cumecs/100km^2/10mm
C_F is PR/100, PR being the percentage runoff
A is the area in km^2
P_F is the design rainfall in mm

The FSR procedure recommends a design storm duration, D_F, expressed as a function of the time to peak of the unit hydrograph, TP, and the annual average rainfall at the site, SAAR

$$D_F = TP(1 + SAAR/1000) \tag{5}$$

By substituting values of SAAR and using Figure 6.64, it will be found that

$Q_F = 0.29 C_F\ i_F\ A$ when SAAR = 600 mm
$Q_F = 0.33 C_F\ i_F\ A$ when SAAR = 1000 mm

where $i_F = P_F/D_F$ the average intensity.

Thus it has been established that by adopting a middle value of the multiples which introduces little error over a wide range of catchment conditions in lowland Britain, two very similar expressions result:

$Q_R = 0.28 C_R\ i_R\ A$ for the rational formula (2)

$Q_F = 0.31 C_F\ i_F\ A$ for the FSR (6)

3. RUNOFF COEFFICIENT

The FSR design recommendations quote a procedure for evaluating C_F (in fact percentage runoff PR) in terms of the catchment wetness and soil type with minor modifying terms (Equation 6.40, p. 420). No formal work has been carried out for evaluating C_R although examples known to the writer use values of a similar order of magnitude to C_F, ie 0.1 to 0.5.

If it can be assumed that identical values for the coefficients of runoff are adopted then the two flood formulae will give results within 10% of each other for the same design storm.

4. RAINFALL RETURN PERIOD

In the use of the rational formula the design return period of rainfall, T_R, is usually carried through to be the return period of the consequent flood. The FSR shows (Section 6.7) that the return period of the flood depends on many factors and in the design procedure selects the rain depth, duration and profile in order to yield, on average, a flood of the desired return period. In fact the depth duration combination that is selected gives a return period, T_F, which varies from 1.6 to 1.8 times the flood return period over a range of common design requirements, 10 to 100 years.

$$\text{Thus } T_F \simeq 1.7\, T_R \tag{7}$$

5. RAINFALL DURATION

Most engineers use the Bransby-Williams formula to obtain the design rainfall duration, D_R, for the rational method

$$D_R = TC = \frac{L}{D} \sqrt[5]{\frac{M^2}{F}}$$

where L is catchment length
D is the diameter of a circle whose area is equal to the catchment area (L/D is a dimensionless circularity factor)
M is catchment area in miles^2
F is the 'fall' or channel slope in ft/100 ft
TC is time of concentration in hour.

A comparison between TC from Equation 6 and TP, the observed time to peak of the 1 hour unit hydrograph, for 80 catchments used in the FSR unit hydrograph study showed TC from small catchments to be much shorter than observed lag or hydrograph rise times. In fact only for larger and steeper catchments does TC approximate observed catchment time characteristics. Bransby-Williams states in his paper, which deals with Indian irrigation and other channels, that "the formula gives a somewhat more rapid concentration than actually takes place in most instances....". As a very approximate guideline for small catchments with a moderate slope:

$$TC = 0.5\ TP$$

which by substitution in Equation 5 gives

$$D_F/D_R = 2 + SAAR/500 \tag{8}$$

6. RAINFALL INTENSITY

The consequences of the difference in return period, Equation 7, and duration, Equation 8, on the intensities i_F and i_R can be obtained from Chapter 2 of Vol. II of the FSR. For small and medium sized lowland catchments, the major difference between i_F and i_R is due to the duration difference, typically, $i_F = 0.38 i_R$. The effect of return period growth factors and areal reduction factor differences are much smaller but have all been combined to give:

$$i_F = 0.44 i_R \tag{9}$$

Note than in such areas the use of the Bilham formula for rainfall intensity will lead to similar values to those of Vol. II of the FSR.

7. COMBINED EFFECTS

By combining Equations (2) and (6) with the result just obtained for intensity differences, Equation (9), it can be seen that for small lowland catchments:

$$Q_F = 0.48Q_R$$

or the flood peak from the FSR is typically about half that obtained from the rational formula. For very small and flat catchments, the inflating effect is much greater and factor of ten differences are possible.

The major source of difference is the design duration and this difference is in turn due to the dominating effect of catchment area in the Bransby-Williams formula. The evidence from UK data is that slope is the primary variable determining response time so that only for larger catchments can the Bransby-Williams formula give reasonable results. For such conditions (area from 200 km^2 to 500 km^2 and S1085 steeper than 5m/km) a repeat of the above exercise has given:

$$Q_F = 1.07Q_R$$

although the margin would be widened if 'Bilham' rainfall were adopted for use in the rational formula because Bilham used mainly lowland raingauges in his analysis.

Flood Studies Supplementary Report No 9 Sept 1979

Short cut to unit hydrograph convolution

1. INTRODUCTION

In the Flood Studies Report (FSR) Vol. 1, a diagram (Figure 6.64 p.466) is presented for direct determination of the peak of the convolution of 75% winter profile with the recommended triangular unit hydrograph. The procedure has become known as the 'curve number' method and is widely used as a short cut to determining the design hydrograph peak. This report expands the description given in the FSR of the principle behind the method, and shows how curve number diagrams may be generated for other unit hydrograph and rainfall profile shapes. The report goes on to demonstrate how the curve number diagram can be simplified, giving a 'Rational' style formula in which the coefficient is composed of the percentage runoff and a routing coefficient. Finally, the concept is extended to the entire hydrograph shape, and a method is presented for obtaining the quick response hydrograph without the need for convolution.

Unfortunately, the original FSR curve number diagram (Figure 6.64) contains small inaccuracies. These inaccuracies are not serious enough to warrant a corrigenda issue to all FSR buyers, but, for reference, a corrected diagram is presented as Figure 1 of this report. The new 'Rational' style formula presented herein, however, renders both new and old curve number diagrams obsolete.

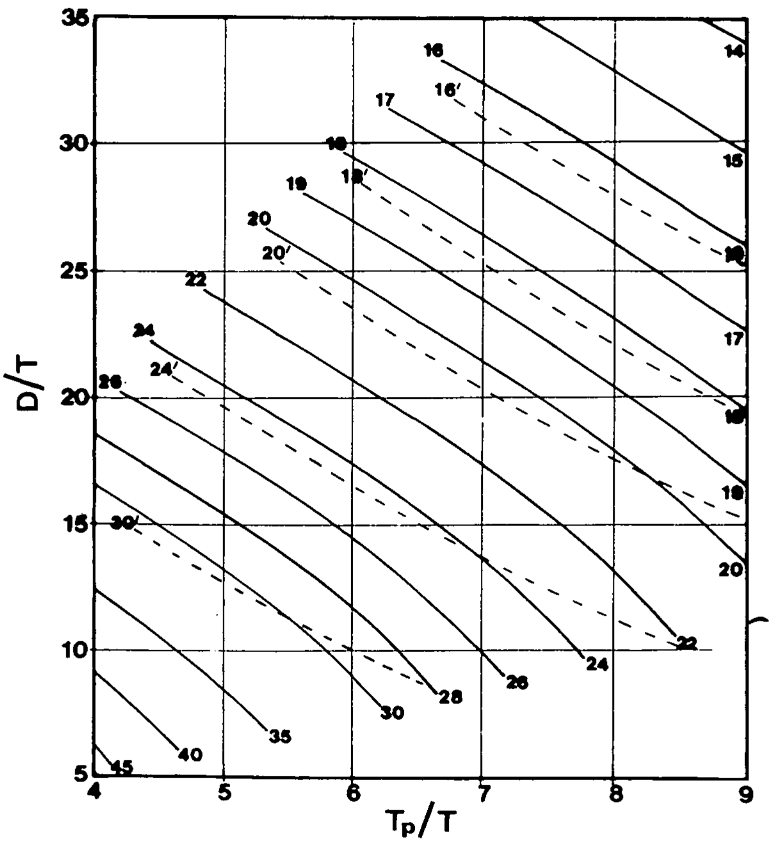


FIGURE 1
Diagram for direct determination of peak of quick response hydrograph

--- original FSR curves
—— corrected curves

2. HOW DOES THE CURVE NUMBER DIAGRAM WORK?

On FSR page 468-9 it states that the curve number diagram is possible because the peak from the convolution of a symmetrical rainfall profile with a simple triangle occurs at the timestep when the peak of the unit hydrograph is multiplied by the peak of the rainfall profile. This is an oversimplistic justification; a curve number diagram is possible for any standardised shape of rainfall profile and unit hydrograph. Moreover, it is not strictly accurate, since with a skewed unit hydrograph, the peak of the convolution does not occur exactly at the timestep when the peak of the unit hydrograph is multiplied by the peak of the rainfall profile. The curve number diagram derives from considering the superposition equations (FSR, Vol 1, p 379). These may be written as the convolution

$$q_t = \sum_{\tau=0}^{t} u_\tau . p_{t-\tau} \qquad (1)$$

where u_τ are successive ordinates at time step T of the T-hour unit hydrograph,

p_τ are depths of effective rainfall in successive periods T

and q_t are ordinates of the quick response runoff hydrograph.

In the FSR it is recommended that

(i) u_τ are scaled from a standardised unit hydrograph, ie

$$u_\tau = (\text{AREA}/\text{T}) .\ f_\tau\{\text{Tp}/\text{T}\} \qquad (2)$$

where AREA is catchment area

Tp is the time to peak of the T hour unit hydrograph

and $f_\tau\{\text{Tp}/\text{T}\}$ are ordinates (depending on the ratio Tp/T) describing a triangle

and (ii) p_τ are scaled from a standardised rainfall profile, ie:

$$p_\tau = \text{PR.P.}\ g_\tau\{\text{D}/\text{T}\} \qquad (3)$$

where PR is percentage runoff,

P is total storm depth occurring in D hours

and g_τ {D/T} are ordinates (depending on the ratio D/T) describing the 75% winter profile

Substituting equations (2) and (3) into equation (1) gives:

$$q_t = \frac{\text{AREA}}{\text{T}}.\text{PR.P} \sum_{\tau=0}^{t} f_\tau\{\text{Tp}/\text{T}\}.g_{t-\tau}\{\text{D}/\text{T}\} \qquad (4)$$

or

$$q_t = \frac{\text{AREA}}{\text{T}}.\text{PR.P}.cn_t \qquad (5)$$

where

$$cn_t = \sum_{t=0}^{t} f_\tau\{\text{Tp}/\text{T}\}.g_{t-\tau}\{\text{D}/\text{T}\} \qquad (6)$$

The original convolution equation (1) has been reduced to a standardised form, equation (6). This may be solved for a range of Tp/T and D/T and the peaks (denoted CN) plotted against Tp/T and D/T. The resulting diagram is the curve number diagram, Fig. 1, which can be used with equation (5) for the direct estimate of the peak of the full convolution. The resulting equation is given in FSR, Vol 1, p467, ie

$$\hat{q} = \frac{CN.AREA.P.PR}{10^5\ T} \tag{7}$$

where the 10^5 arises from consideration of units.

From the above development it is clear that a curve number diagram could be drawn for other assumptions of the form of f_τ and g_τ, and the approach is not restricted to simple triangles and symmetrical profiles. If, however, f_τ or g_τ depend on other factors as well as on Tp/T and D/T (as, for example when loss rate separation of effective rainfall is used, and the effective rainfall profile shape varies with the ratio of loss to total rainfall), additional dimensions will be necessary to define CN.

A result of particular interest arises when f_τ and g_τ are both rectangular, having time bases TB and D respectively. The curve number contours degenerate to a series of rectangles given by:

$$CN = (.28\ T/D).10^3 \quad \text{for } D > TB \tag{8}$$

$$CN = (.28\ T/TB).10^3 \quad \text{for } TB > D \tag{9}$$

Putting D = TB and substituting for CN in equation (7) gives the Rational formula

$$\hat{q} = .28\ (\frac{PR}{100}).(\frac{P}{D}).AREA \tag{10}$$

This result is discussed in detail in Supplementary Report No.8. The similarity between equation (7) and the Rational Formula is explored again below.

3. SIMPLIFICATION OF THE CURVE NUMBER APPROACH - A "RATIONAL" FORMULATION

Examination of equation (7) suggests that, since $\hat{q}$ should be independent of the solution timestep, the quotient CN/T should be constant for any given Tp and D. It follows that if Tp/T and D/T are both increased by some ratio, CN is reduced by the same ratio. Consequently, along lines of equal D/Tp on the curve number diagram, the ratios CN.(Tp/T) and CN.(D/T) should be constant. The curve numbers of Figure 6.64 and Figure 1 do indeed exhibit this property, though the property for the summation case of equation (6) is really only approximate, small errors arising due to the finite difference representation of f_τ and g_τ, and also from the necessity to change Tp with changing T. The property is exact when the summation is replaced by the convolution integral.

Because of this property, the curve number diagram can be reduced to a plot of either CN.Tp/T or CN.D/T against D/Tp. If CN.D/T is plotted it results in a simple 'Rational' formulation, ie writing

$$RC = \frac{CN.D}{T.10^3} \tag{11}$$

and substituting RC into equation (7) gives

$$\hat{q} = RC.(\frac{PR}{100}).(\frac{P}{D}).\ AREA \tag{12}$$

where the rational coefficient has been split into a routing coefficient, RC, and the volume of runoff coefficient PR/100. A plot of RC against D/Tp is given in Figure 2, and this diagram may be used together with equation (12) for the direct determination of the peak of the full convolution. As stated earlier, the property CN.D/T = constant for D/Tp constant is only approximate. However, the variation about the line in Figure 2 for Tp/T between 4 and 9 is less than ½%.

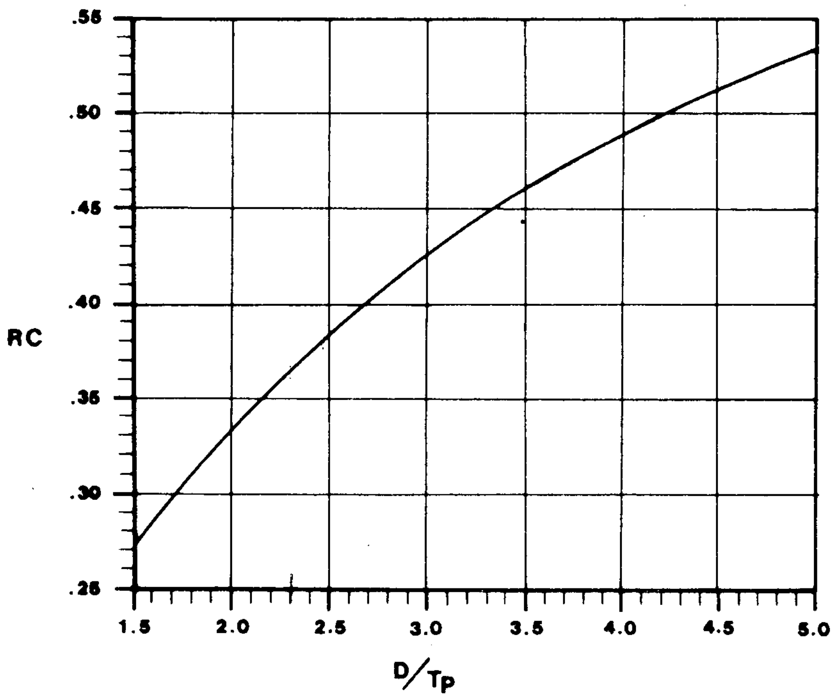


FIGURE 2

Graph of routing coefficient for use in "Rational" formula

(based on triangular unit hydrograph and 75% winter rainfall profile)

4. TIME OF THE PEAK FLOW

As stated in Section 2, it is not strictly accurate that the peak of the convolution of a symmetrical rainfall profile with a simple triangle occurs when the peak of the unit hydrograph is multiplied by the peak of the rainfall profile (ie at timestep Tp + (D-T)/2). In fact, the peak only occurs at that timestep when

(i) both rainfall profile and the unit hydrograph are symmetrical

or (ii) rainfall profile and unit hydrograph are of the same shape and time base, but mirror images of each other

Furthermore if the unit hydrograph peak was 'missed' by the timestep, the exact peak would not be provided by the convolution summation but would need to be interpolated.

With a symmetrical rainfall profile, but a unit hydrograph skewed to the left, it is clear that the peak of the convolved hydrograph will occur later than Tp + (D-T)/2. The question is by how much. With a rectangular profile of duration D(< 2Tp) and the FSR triangular unit hydrograph it is possible to show theoretically that the peak is delayed by about 0.1D. With a peaked rainfall profile, the delay is less, but is difficult to quantify, especially with the use of a discrete timestep. Based on trial convolutions, the following three part relationship was derived for rise time, TR:

$$D < 2.2\ Tp, \quad TR = Tp + (D-T)/2 + .043D \tag{13}$$

$$2.2\ Tp < D < 3\ Tp, \quad TR = Tp + (D-T)/2 + .095\ Tp \tag{14}$$

$$D > 3\ Tp, \quad TR = Tp + (D-T)/2 + .095\ Tp + .02\ (D-3\ Tp) \tag{15}$$

As can be seen, the departure from Tp + (D-T)/2 is generally small, and, if $Tp/T \approx 5$, rarely exceeds a whole time step.

5. DESIGN HYDROGRAPH SHAPE

This report has so far been concerned with the hydrograph peak. However, the standardised convolution (equation (6)) could equally be evaluated for any point in the convolution and a diagram similar to Figure 2 prepared for that point. Thus complete hydrograph shape, and not just peak, is a function of D/Tp. Figure 3 illustrates the range of hydrograph shapes obtained for 1.4 < D/Tp < 5.0. As expected, when D is relatively short compared to Tp, hydrograph shape is more skewed, resembling the unit hydrograph, whereas, when D is longer, hydrograph shape tends more towards the rainfall profile. Figure 3 can be used as a short cut to deriving the complete design hydrograph, sketching in a curve for the exact value of D/Tp required, multiplying all time abscissae by Tp and all flow ordinated by $\hat{q}$.

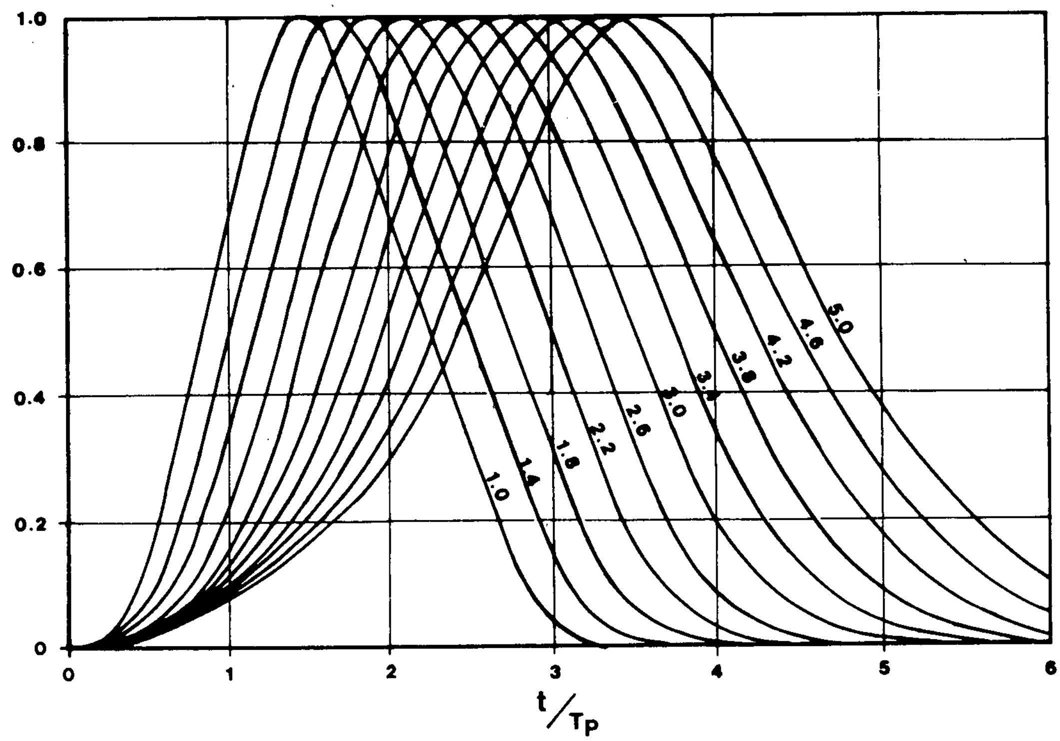


FIGURE 3 Standard hydrograph shapes for stated values of D/Tp

In fact, for the discrete convolution of equation (6), hydrograph shape also depends to some extent on the solution timestep, T. Note, for example, from equations (13) to (15):

$$TR/Tp \approx 1 + \tfrac{1}{2}(D/Tp - T/Tp) \tag{16}$$

Figure 3 is drawn for T = T/5; from equation (16) the effect of using T = Tp/9 would be to increase TR/Tp by .044; This shows the effect of T on hydrograph shape is quite small. However, if desired, the effect of T could be taken into account by using equations (13) to (15) to locate the position of the peak of the design hydrograph in Figure 3 and then sketching in a corresponding hydrograph shape.

6. EXAMPLE OF QUICK SOLUTION TO CONVOLUTION PROBLEM

To demonstrate the use of the quick solution to the convolution problem steps 15 to 18 of the worked example given in FSR Vol 1, pp 466-468 have been reworked.

$$
\begin{array}{llll}
\text{given} & \text{AREA} & = & 63.2 \text{ km}^2 \\
 & T & = & .5 \text{ hours} \\
 & Tp & = & 1.9 \text{ hours} \\
 & D & = & 4.5 \text{ hours} \\
 & P & = & 98.3 \text{ mm} \\
 & PR & = & 47.2\% \\
\text{then} & D/Tp & = & 2.37 \\
\text{from Fig 2} & RC & = & .375 \\
\text{and} & \hat{q} & = & .375 \left(\frac{47.2}{100}\right) . \frac{98.3}{4.5} . 63.2 \\
 & & = & 244.4
\end{array}
$$

this result is 1.4% larger than that given in steps 15 and 18 of the FSR example, but is based on an interpolation to find the exact peak.

To find the complete hydrograph, a curve for D/Tp = 2.37 was sketched on fig 3 and ordinates at intervals of T/Tp (=.263) abstracted, and multiplied by $\hat{q}$, giving

T/Tp	.263	.526	.789	1.052	1.315	1.578	1.841	2.104	2.367	2.630	...
$q/\hat{q}$	.005	.030	.100	.210	.410	.625	.860	.980	.970	.840	...
q	1.2	7.3	24.4	51.3	100.2	152.8	210.2	239.5	237.1	205.3	...

Comparison of the q above with the bottom line of FSR table 6.24 shows a good fit to the full convolution, the quick solution consistently lagging slightly behind the full convolution. The quick solution is however based on a sketched line and must be subject to errors of interpolation; some discrepancy must be expected.

7. CONCLUSIONS

A quick 'Rational' style procedure is presented for the direct determination of the peak of the convolution of the FSR triangular unit hydrograph with the 75% winter rainfall profile:

$$\hat{q} = RC . \left(\frac{PR}{100}\right) . \frac{P}{D} . \text{AREA}$$

where RC is defined from D/Tp in Fig 2. Complete hydrograph shape may also be estimated from D/Tp using Figure 3, multiplying all time abscissae by Tp and all flow ordinates by $\hat{q}$. Account may be taken of the effect of T on hydrograph shape using equations presented for rise time - equations (13), (14) & (15).

A guide to spillway flood calculation for a cascade of reservoirs

1. INTRODUCTION

This supplementary report[+] is rather different from others in the series in that it is an extension of the ICE guide to floods and reservoir safety (Ref 1) rather than a supplement to the FSR itself.

Although a procedure is given in the FSR for estimation of a maximum flood hydrograph (FSR I.6.8.3) the method provides only an input to spillway design calculations rather than a blueprint for them. Following deliberations of its Working Party on Floods and Reservoir Safety, the Institution of Civil Engineers published an engineering guide entitled "Floods and Reservoir Safety" (Ref 1 - here referred to as the ICE Guide). This places the FSR methodology in the context of reservoir spillway design, presenting standards for combining a design flood inflow to a reservoir with a suitable design value of initial level and allowances for wave surcharge (Table 1 of ICE Guide).

In general, the ICE Guide extends rather than revises the FSR recommendations. There are, however, some differences of which those relevant to this supplementary report are:

- the FSR's 'estimated' maximum flood (EMF) becomes, without change in meaning, the 'probable' maximum flood (PMF).
- the ICE Guide recommends the distinction of summer and winter values of probable maximum precipitation (PMP) and that snowmelt allowances need only be added to winter rainfall. This follows a refinement of the FSR procedure developed, with Meteorological Office approval, jointly by the ICE Working Party and the Institute of Hydrology.
- the ICE Guide includes specially prepared maps (Figs 5-7 of Guide) which provide point values of RSMD as distinct from the areal values required in the FSR methodology. Although intrinsically less accurate, the maps provide an acceptable short-cut to calculation of RSMD if appropriate areal reduction factors are applied.
- The ICE Guide states that on some very large reservoired catchments, where the duration of the design storm is several days, it is inappropriate to assume a symmetrical storm profile. In such cases the recommendation is to adopt the profile of the severest sequence of storms of the required duration that has been observed locally.

The ICE Guide provides a brief outline (p. 34) of how maximum flood estimation might be applied to a catchment which contains a cascade of reservoirs. However, experience has shown that the single storm approach, outlined in the ICE Guide, makes it difficult to preserve the flows between individual subcatchments in appropriate time sequence and is at least as time-consuming as the method given here. Moreover, the ICE Guide concentrates on PMF estimation whereas its design standards sometimes require T-year flood estimation. This supplementary report sets out procedures for both T-year and PMF calculations for a cascade of reservoirs. A worked example of the PMF case is presented alongside the step-by-step description.

[+] *This report was prepared by D F MacDonald in cooperation with colleagues at Binnie and Partners (London) and with assistance from the Institute of Hydrology (D W Reed and M J Lowing).*

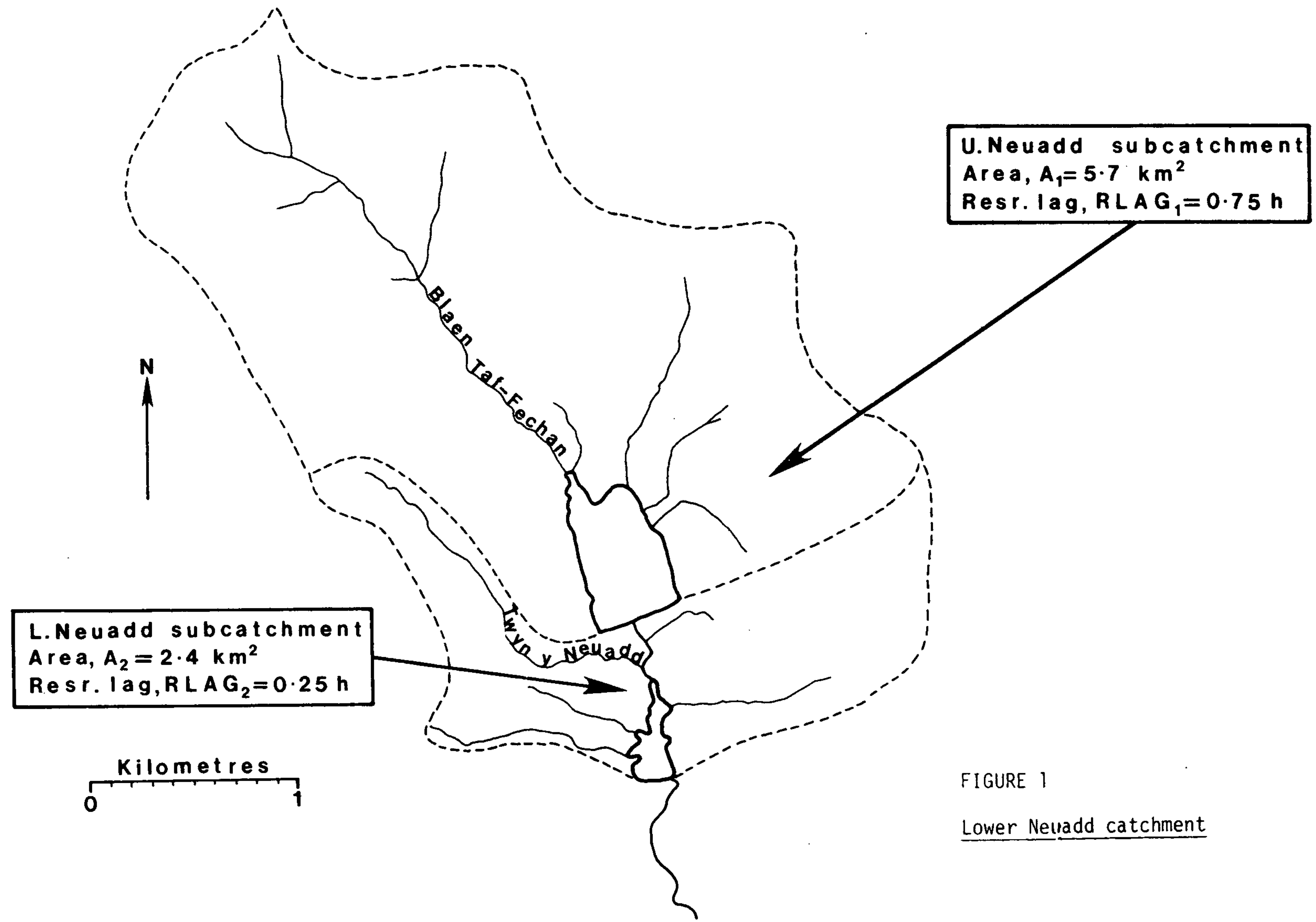


FIGURE 1

Lower Neuadd catchment

2. THE RECOMMENDED PROCEDURE

In addition to the usual FSR nomenclature, the following terms are used:

- N — The number of reservoirs in the cascade above (and including) the one being checked. Reservoirs in the cascade are numbered 1 to N by altitude, starting at the head of the catchment.
- A_i — Area of direct (or local) subcatchment (km^2) draining to reservoir i (i.e. without passing through other reservoirs).
- $RLAG_i$ — Reservoir lag (hours) imposed on runoff from areas A_i due to routing through reservoir i (defined as the time between peak inflow and peak outflow).
- $CLAG_i$ — Cumulative lag (hours) imposed on runoff from area A_i due to routing through one or more reservoirs downstream to, and through, reservoir N.
- MRLAG — Mean reservoir lag - an areally weighted average of the cumulative lags. It is a measure of the mean lag (hours) imposed on runoff from the entire catchment by the routing effects of the N reservoirs.

The steps set out below show how to derive the flood inflow hydrograph for the bottom reservoir of a cascade. Intermediate reservoirs in the cascade are covered by successively treating them in separate calculations as bottom reservoirs of smaller cascades exactly in the manner described below. The top reservoir is checked by a "single reservoir" analysis. The worked example, included by kind permission of the Welsh Water Authority, relates to the Upper and Lower Neuadd reservoirs and illustrates the calculation of summer PMF for the lower reservoir. The catchment is shown in Fig. 1; it lies in the Brecon Beacons National Park about 12 km north of Merthyr Tydfil.

For the T-year event	*For the PMF event (where different)*	*Example*
Establish design storm duration		
(a) Determine Tp of the 1 hour unit hydrograph for the entire catchment of the bottom reservoir according to FSR step 6 (460, 21)*		MSL is measured to the point of inflow to Lower Neuadd. The S1085 calculation uses pre-inundation levels for Upper Neuadd.
$Tp = 46.6\ MSL^{0.14}\ S1085^{-0.38}\ (1+URBAN)^{-1.99}\ RSMD^{-0.4}$	The Tp value so obtained should be reduced by a third (FSR I.6.8.3, p. 470). The effect is to increase the unit hydrograph ordinates by 50% compared to the T-year case.	
MSL is the main stream length to the bottom reservoir (see Box 1)		MSL = 3.62 km
S1085 is the stream slope between 10% and 85% points going up the main stream (see Box 2)		S1085 = 59.5 m/km
URBAN is the fraction of the catchment in urban development		URBAN = 0.0
RSMD may be calculated from FSR step 3 or from maps in the ICE Guide (but see Box 3)		RSMD = 86 mm
		Tp (from equation) = 1.99 hours
		(× 2/3 in PMF case) = 1.33 hours

* *References to FSR procedural step numbers apply equally to the original report (Ref 2) and the subsequent guide (Ref 3); the corresponding page numbers are given in brackets.*

BOX 1

The main stream length (MSL) is likely to extend from the headwaters of the catchment through one or more of the reservoirs in the cascade. MSL is measured either to the bottom embankment or to the point of inflow to the bottom reservoir. The first approach can be adopted where the bottom reservoir is small and calculations are carried out manually. The latter method is appropriate when rainfall on the reservoir surface is considered separately. This is the required approach when the surface area of the bottom reservoir exceeds 5% of the total catchment area, and is also the approach commonly adopted where unit hydrograph convolutions are carried out by computer. Irrespective of the approach used to determine the inflow hydrograph, the flows must be routed through the reservoir.

BOX 2

Where MSL is measured to the bottom reservoir embankment or where the main stream passes through another reservoir it may be necessary to take pre-inundation levels when computing S1085, to ensure that a representative value is obtained. To calculate Tp for any catchment, MSL and S1085 would normally be taken from the stream draining the largest area. In some cases, particularly where a reservoir extends well up the catchment, there may be several streams of similar sizes. In these situations the Tp calculation should be based on a typical stream draining to the reservoir.

BOX 3

When using the RSMD maps (Figs 5-7 in ICE Guide - Ref 1) for large catchments, RSMD is calculated by averaging the values at grid intersection points and then applying an areal reduction factor. A reduction of 3% is recommended for a 10 km^2 catchment, increasing to 7% over 100 km^2 (Ref 4).

For the T-year event	*For the PMF event (where different)*	*Example*
(b) Calculate the Tp value for the 'direct subcatchment' (see Box 4) of each reservoir in the cascade from the equation given in (a) and choose a convenient time interval (T) such that $T \simeq Tp/5$ for the shortest Tp of any of the subcatchments. Use this time interval in all subsequent calculations. Avoid adopting an unnecessarily short time interval.	Multiply the calculated Tp values by 2/3 before choosing, and adjusting for, time interval.	Multiplying catchment characteristic estimates of Tp by 2/3: For Upper Neuadd, Tp = 0.99 hours For Lower Neuadd, Tp = 0.94 hours (Twyn y Neuadd taken as main stream) A value of T = 0.25 hours was adopted.
If necessary, adjust the Tp values, for the subcatchments as well as for the entire catchment, to allow for the change in data interval from 1 hour. (FSR step 8; 462, 22). new Tp = old Tp + (T - 1)/2		The adjusted Tp values are: For entire catchment: 0.95 hours For Upper Neuadd: 0.62 hours For Lower Neuadd: 0.57 hours

BOX 4

Direct subcatchment means the natural drainage area below one dam and above the next. The entire land area between the two adjacent dams is treated as a single subcatchment, even if it involves separate side streams discharging into the bottom reservoir.

For the T-year event	For the PMF event (where different)	Example

(c) Determine the mean reservoir lag, MRLAG.

: measure subcatchment areas, A_i
: estimate reservoir lags, $RLAG_i$
: compute cumulative lags for each subcatchment, $CLAG_i$
: determine MRLAG as areally-weighted average of cumulative lags.

See Box 5 for details.

$$MRLAG = \sum_{i=1}^{N}(A_i CLAG_i) \Big/ \sum_{i=1}^{N} A_i$$

Example

	i	A_i (km^2)	$RLAG_i$ (hours)	$CLAG_i$ (hours)
U. Neuadd	1	5.7	0.75	1.0 (=0.75+0.25)
L. Neuadd	2	2.4	0.25	0.25

$$MRLAG = (5.7 \times 1.0 + 2.4 \times 0.25)/8.1$$

$$= 0.78 \text{ hours}$$

BOX 5

It is important to extend the design storm duration not just for the lag effect of the reservoir under study but also for that of any upstream reservoir. Many configurations of reservoirs and their associated subcatchments are possible. The following procedure is intended to define a consistent method of evaluating mean reservoir lag (and, hence, design storm duration) for most configurations likely to arise.

Procedure for evaluating MRLAG

1. Focus attention on the reservoir being checked. Mean reservoir lag, MRLAG, is never less than the lag of this reservoir alone.

2. Number by altitude those reservoirs affecting runoff at the design site: that with the highest spillweir crest AOD being numbered 1. Reservoir N is, of course, the reservoir being checked. Reservoirs not on the main stream but which nevertheless discharge into the reservoir being checked should be included in the numbering system.

3. Evaluate area of direct subcatchment to each reservoir. The direct (or local) subcatchment area to the ith reservoir, A_i, is defined as the area draining directly to the reservoir (as opposed to through or past another reservoir).

4. From reservoir storage/outflow characteristics, or from previous calculations (see (m) below), estimate the lag, $RLAG_i$, imposed by each reservoir. (A measure of reservoir lag time (secs) is given by the mean slope of the line relating storage (m^3) to outflow ($m^3\ s^{-1}$).)

5. Consider each subcatchment area, A_i, in turn. Evaluate the cumulative lag, $CLAG_i$, imposed on runoff from that area. $CLAG_i$ is taken as the sum of the individual reservoir lags, $RLAG_i$, for all reservoirs in the cascade from the ith to the Nth. (Consider only reservoirs which are on the same tributary as A_i, i.e. which affect the passage of water from A_i).

6. Evaluate the mean reservoir lag to site N, MRLAG, by taking an areally weighted average of the cumulative lags for each local catchment area. Thus:

$$MRLAG \doteq \sum_{i=1}^{N} (A_i\ CLAG_i) \Big/ \sum_{i=1}^{N} A_i$$

7. Note that whereas for PMF calculations a conservative (i.e. too large) value of reservoir lag can be adopted without fear of it leading to an inadequate flood intensity, this does not apply to T year calculations where too small or too large a value of reservoir lag (and hence design storm duration) may inadvertently bias the implied probability.

For the T-year event	*For the PMF event (where different)*	*Example*
(d) Determine design storm duration from $D = (1 + \frac{SAAR}{1000})(Tp + MRLAG)$		SAAR = 2200 mm $D = 3.2 \times (0.95 + 0.78)$ $= 5.53$ hours
SAAR is the 1941-70 average annual rainfall over the total catchment area to the bottom reservoir.		
The Tp used here is the adjusted value for the entire catchment to reservoir N taken from (b) above.		
Round the storm duration to the nearest odd integer multiple of the time interval.	The duration should always be rounded up to the next odd integer multiple of the time interval.	Rounded to D = 5.75 hours

Design storm rainfall and runoff

(e) Calculate the design storm rainfall, reduced for the entire catchment area, as in FSR steps 10 and 11 (462, 23). Snowmelt calculations are not involved. Note that the FSR examples refer to durations up to 48 hours. For longer durations refer to FSR Vol II.	Follow FSR steps 10M and 11M (472, 32). Table 3 of the ICE Guide lists the seasonal variation in PMP as a function of SAAR. If there is doubt as to whether the critical design event will result from a summer or winter storm, carry out parallel calculations for summer and winter cases (see Box 6). The appropriate snowmelt allowance must be added to the winter design storm rainfall.	For D = 5.75 hours and AREA = 8.1 km^2, summer PMP, P = 224 mm

BOX 6

In the PMF case, the critical storm will be either the summer or the winter event depending upon which produces the maximum water level when wave surcharge is added to flood rise. It is often possible to determine this by comparing the difference between summer and winter wave surcharge values with that between the flood rises that result from summer and winter events on the total catchment to the bottom reservoir, if all the effects of upstream reservoirs are ignored.

(f) Calculate percentage runoff (PR) for the design storm over the total catchment as in FSR steps 12, 13 and 14 (465, 24). Frozen ground is not considered.	Follow FSR step 12M (474, 34). For a winter design storm the possibility that the catchment surface could be frozen should be considered. The recommended allowance for frozen ground is that the soil index be set to a value of 0.5 (472, 31).	PR = 85%

For the T-year event	*For the PMF event (where different)*	*Example*
(g) Distribute the total catchment design storm rainfall excess according to the 75% winter profile as in FSR step 16 (467, 26).	Distribute the design rainfall in a symmetrical nested profile such that the PMP in every duration is centred on the peak of the profile. See step 16M (475, 35). For storm durations exceeding 2 days, see Box 7.	

BOX 7

In the PMF case, should the design storm duration exceed two days, it is recommended that the rainfall profile observed during the severest sequence of storms of the required duration that has been measured locally be used in preference to the standard symmetrical profile. Note that it is the profile - i.e. the proportional distribution - that is taken from local information not the rainfall depths themselves.

Compute reservoir inflow hydrographs

(h) Calculate the 'average non-separated flow' (ANSF) for the total catchment as in FSR step 19 (469, 30) and apportion the value so obtained between the individual subcatchments based on their respective areas. See also Box 8.	Follow FSR step 19 as in T-year case but remember that the CWI value to be used comes from step 12M (474, 34).	ANSF = 0.7 m^3/s (total catchment) (0.49 m^3/s from Upper Neuadd, 0.21 m^3/s from Lower Neuadd).

BOX 8

The ICE Guide recommends (p 35) the use of long term mean daily flow rather than ANSF. As any difference between these two flows will not be material in its impact on the flood routing, either term may be adopted.

(i) Calculate the direct inflow to each reservoir in the cascade by convolving (FSR step 18; 468, 28) the total catchment design storm rainfall excess from (g) with the unit hydrograph for the direct subcatchment of each reservoir and then adding the appropriate value of ANSF from (h). Appropriate allowance may also have to be made at this stage for any flows to a bywash or from a catchwater.	As in the T-year case but use the PMF (peakier) unit hydrographs.	The Upper Neuadd and Lower Neuadd subcatchment unit hydrographs were determined from the Tp values of 0.62 and 0.57 hours from (b). A catchwater feeding Upper Neuadd was assumed blocked.

For the T-year event	*For the PMF event (where different)*	*Example*
Route, lag and add hydrographs		

BOX 9

Alternative levels may be adopted in certain cases where reservoir control procedures require, and discharge capacities permit, operation at or below specified levels throughout the year.

(j) Route the inflow hydrograph through the top reservoir of the cascade, remembering to add in rain falling on the reservoir surface if this has been treated separately (see Box 1). Determine the routed outflow. For dams which fall into Categories B or C as defined by the ICE Guide, the reservoir is assumed to be just full (i.e. no spill) at the beginning of the design event. For those dams classified as Category A or D the reservoir is assumed to be full and spilling an amount equivalent to ANSF in (h). See also Box 9.		Flood inflow hydrograph routed through Upper Neuadd reservoir. See Fig 2 (a).
(k) Preserving the correct time sequence (see Box 10), add the routed outflow from the top reservoir to the direct runoff from the subcatchment of the next reservoir in the cascade to produce the total inflow hydrograph which is routed through the second reservoir of the cascade. In computing the inflow hydrograph to the second reservoir due account must be taken of bywash and/or catchwater flows.		Ordinates of routed Upper Neuadd outflow added to Lower Neuadd direct subcatchment PMF hydrograph. See Fig 2(b). No lag or attenuation was applied to Upper Neuadd outflows as only 350 m separates the two reservoirs.

BOX 10

If the distance between two adjacent reservoirs in the cascade is such that the routed outflows from the upper reservoir are likely to take one or more time intervals to travel to the lower reservoir then the routed outflows must be appropriately lagged before being added to the runoff hydrograph of the direct subcatchment to the lower reservoir. In a few cases the translation (time delay) from one reservoir to the next may be accompanied by significant attenuation of the hydrograph. If so, river channel routing may be needed.

(l) The previous step (k) is repeated down the valley so that the inflow hydrograph to the bottom reservoir of the cascade embodies runoff from the total catchment area. This hydrograph is then used to test the adequacy of the spillway arrangements of the bottom reservoir.		Total Lower Neuadd inflow hydrograph from (k) routed through Lower Neuadd.
(m) In the light of the total cascade calculation, review the original assumptions made about the individual reservoir lags, $RLAG_i$, in the derivation of MRLAG in (c). If the reservoir lags are found to have been significantly under- or over-estimated then the calculation may need to be repeated from (c) onwards.	If the lags are found to be slightly shorter than estimated, a repeat calculation may not be justified as it would result in a somewhat smaller flood.	Reservoir lags derived from routing calculations compared with those estimated in (c). No revision was necessary.

3. DISCUSSION

The approach detailed above for T-year and PMF events has been used for the statutory inspections of the reservoirs in more than ten cascades in northern England and Wales. The results produced are thought to be reasonable by all parties concerned with the investigations, but are probably incapable of proof. However, following the approach will ensure consistency of standards.

4. REFERENCES

1. Institution of Civil Engineers, Floods Working Party. *Floods and Reservoir Safety: an Engineering Guide.* ICE, London, 1978.

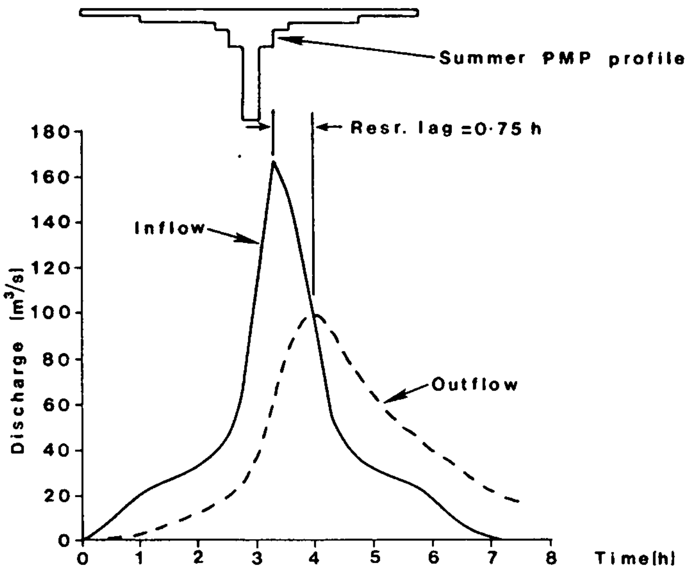
2. Natural Environment Research Council. *Flood Studies Report.* NERC, London, 1975 (five volumes).

3. Institute of Hydrology. Guide to the Flood Studies Report. *IH Report No. 49, 1978.*

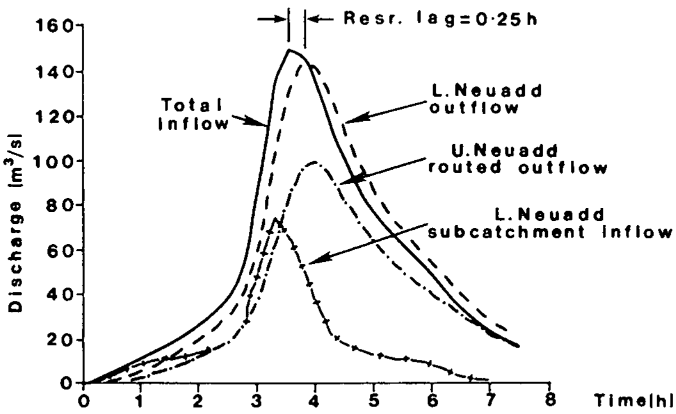
4. Farquharson, F A K et al. Some aspects of design flood estimation. Inspection, Operation and Improvement of Existing Dams. *Proc. Symp. Newcastle University/BNCOLD, 1975.*

FIGURE 2

Lower Neuadd summer PMF



(a) Routing through Upper Neuadd reservoir



(b) Routing through Lower Neuadd reservoir

Flood Studies Supplementary Report No 11 April 1983

A conversion factor for stream frequency derived from Second Series 1:25,000 scale maps

1. INTRODUCTION

Stream frequency (STMFRQ) is derived by counting the number of stream junctions in a catchment and dividing by the catchment area in square kilometres (FSR section I.4.2.2.). STMFRQ values used in the FSR for Scotland, Wales and England are based mainly on the First (Provisional) edition of the 1:25,000 topographic map series. These First Series maps are under active revision by the Ordnance Survey and the new Second Series maps have been found to show more streams than the corresponding First Series maps. STMFRQ calculated from a Second Series map will result in an over-estimate of the mean annual flood when using the recommended FSR equations (Figure I.4.15). This report outlines the history of the 1:25,000 map series and provides a conversion factor relating First and Second Series junction counts. More details of the method of comparison are given in Institute of Hydrology Report No 84.

In Ireland 1:25,000 scale maps are not generally available and the interested reader should seek guidance on STMFRQ calculations from the Department of Finance (Northern Ireland) or the Office of Public Works (Republic of Ireland).

2. HISTORY OF THE 1:25,000 MAP SERIES

The First Series maps are based on the nineteenth century 1:10,560 scale County Series, revised in places according to more recent data. Since 1965 the Ordnance Survey have been producing Second Series maps. About 60% of these maps have been based on post-war survey and revision. Some of these have been produced by revising the original County Series (in which case the river network information on the different series should compare closely) whilst others have been based on a more stringent specification for surveying new areas (in which case significant differences between First and Second Series may arise). The remaining 40% of the Second Series maps have been based on aerial photography and as a result the river network information will differ from the original First Series and may not be entirely consistent with the other Second Series maps. Unfortunately, it is difficult to identify the mapping practices for all the First and Second Series maps and hence it is not possible to incorporate this background information into an analysis.

3. COMPARISON OF FIRST AND SECOND SERIES MAPS

Figure 1 shows the location of the First and Second Series maps which were available for comparison. Stream junctions were counted on each map in a 20 cm × 20 cm square (equivalent to 25 km^2 on the ground) positioned wherever possible in the north-west corner of each pair of maps. Figure 2 shows that the junction counts on the Second Series are generally higher than those on First Series maps and that there is considerable scatter in the relationship when the number of counts is high. Two options were considered. The first was to adopt a 'national' conversion equation which would be simple to apply, the second was to use a number of regional conversion equations. In order to test whether the second option led to improved estimation a subset of 50 maps, ten from each of five locally homogeneous areas, was selected for detailed study. These map sheets are indicated by an asterisk in Figure 1.

Local regressions of First Series (S1) on Second Series (S2) junction counts did not significantly reduce the error of estimation compared with fitting a national regression equation to the complete data set (IH Report No 84). A second experiment tested whether the policy of the Ordnance Survey to use aerial surveys in more remote upland areas might be reflected in an altitude-dependent relationship between S1 and S2. However, the inclusions of an altitude term in the 'regional' equation offered no improvement in the error of estimating First Series junction counts. Thus a single regression equation relating S1 to S2 junction counts is appropriate. Equation 1 shows the structural relation derived by minimising the orthogonal sum of squares with two outliers excluded.

$$S1 = 0.05 + 0.74\ S2 \qquad r^2 = 0.80 \qquad (1)$$

For practical purposes it is appropriate to estimate First Series junction counts from

$$S1 = 0.74\ S2 \qquad (2)$$

When both First and Second Series maps can be obtained, it is preferable to use the First Series, and follow the FSR procedure described in section I.4.2.2.

4. EXAMPLE

Of the three 1:25,000 sheets which cover the Megget at Henderland catchment (in south-east Scotland) two are available in the First Series (NT11 and NT12) and one as a Second Series map (NT22). The calculation of STMFRQ is as follows:

Number of First Series junctions within the catchment on NT11 & NT12	= 35 + 113 = 148
Number of Second Series junctions within the catchment on NT22	= 69
Adjusted Second Series total (0.74 x 69)	= 51
Catchment area	= 56.7 km^2
STMFRQ = (148 + 51)/56.7	= 3.51

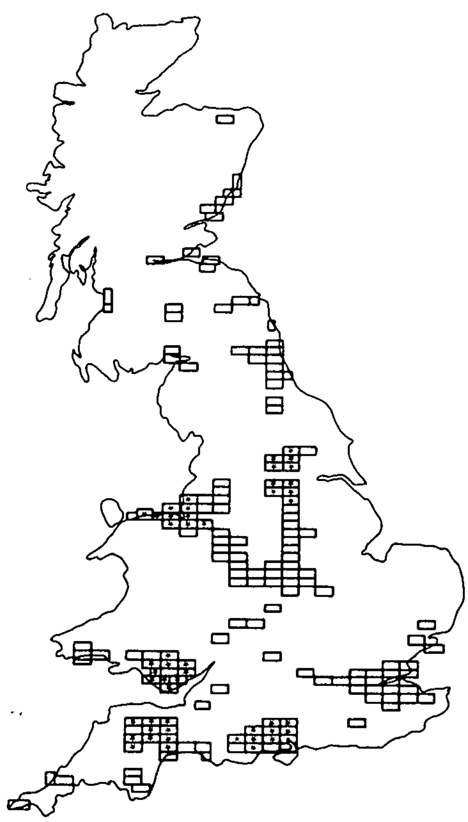


FIGURE 1 Distribution of 1:25,000 map pairs

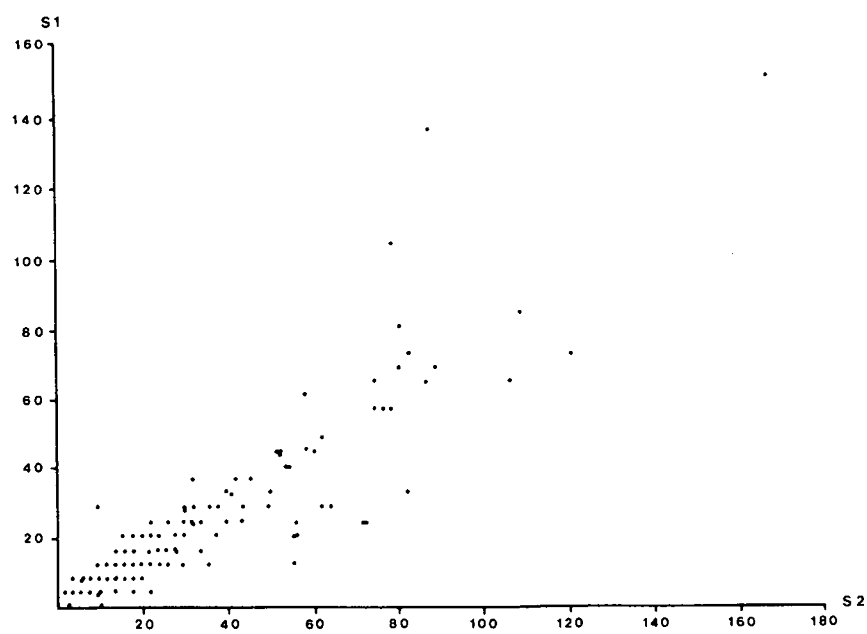


FIGURE 2 Scatter diagram showing relation between sample First and Second Series junction counts

Flood Studies Supplementary Report No 12 April 1983

Assessing the return period of a notable flood

1. INTRODUCTION

It is sometimes useful to estimate the return period of a notable flood that has occurred on an ungauged catchment. The most obvious example is where the flood has caused damage and could, with knowledge of its return period, provide a good guide in assessing the benefit of river improvement works. The essence of the problem is to deduce the rarity of the flood from whatever information can be gathered about the causal rainstorm.

Prior to publication of the FSR, the problem was most often tackled by estimating the return period of the storm rainfall and equating this to the return period of the resultant flood. The approach provides a first approximation but can give misleading results if, for example, the storm occurred on an exceptionally dry catchment. A spurious result could also arise if the duration of the storm was wildly different from that which is normally 'critical' to flooding at the site in question. Other features of the rainstorm - for example, its temporal distribution (or profile) - can also affect the resultant flood but are ignored in the simple approach.

This supplementary report shows how the problem can be tackled using the rainfall/runoff method of flood estimation developed in Vol I, Ch 6 of the FSR. The procedure is lengthy but relatively straightforward. It encourages the user to seek out detailed information about the causal rainstorm but is also applicable when only the rainfall depth is known. A complementary approach, not considered here, is to estimate peak flow from wrack mark evidence† and to compare this with a flood frequency curve developed from catchment characteristics. Other relevant references are I.2.8 and IV.4 which describe the use of historical flood marks in flood estimation.

2. PROCEDURE

Assessment of the flood return period of a notable event is carried out in three stages for which Figure 1 provides a key.

The first stage is to apply the FSR rainfall/runoff method in the 'no data' case. Catchment and climate characteristics are obtained from maps and the usual procedure (Vol I, Sect 6.8.2) followed to synthesise flood peaks for a range of return periods. These values are plotted on Gumbel paper and a flood frequency relationship sketched in (Stage 1 in Figure 1). (Note that the convolution process can be avoided by the short cut described in Supplementary Report No 9.) The above represents the normal design use of the unit hydrograph/losses model, i.e. to produce a flood peak of specified rarity from standard 'design' inputs. Figure 2 shows the routes by which these inputs influence the flood peak generated by the model.

The second stage of the procedure is to apply the unit hydrograph/losses model to the actual storm event. To do this it is first necessary to determine the depth, duration and profile of the storm, and the antecedent catchment wetness. Advice on

†*Dalrymple, T. and Benson, M. A. "Measurement of peak discharge by the slope area method." Techniques of water resource investigations of the United States Geological Survey. Book 3 Applications of Hydraulics, Chapter A2. 1967*

how to gather this information from meteorological records (or recollections) of the notable event is given in Section 3. Using these 'real' inputs in place of the design inputs, the unit hydrograph/losses model (Figure 2) is applied again, this time to simulate the flood arising from the notable storm. Note that it is generally necessary to construct the triangular unit hydrograph so that a full convolution can be carried out. (The short cut to convolution applies only to use of a standard design storm profile.)

In the final stage, the peak of the simulated hydrograph for the event is entered on flood frequency relationship derived in Stage 1. The required return period is then read off. (Stage 3 in Figure 1.)

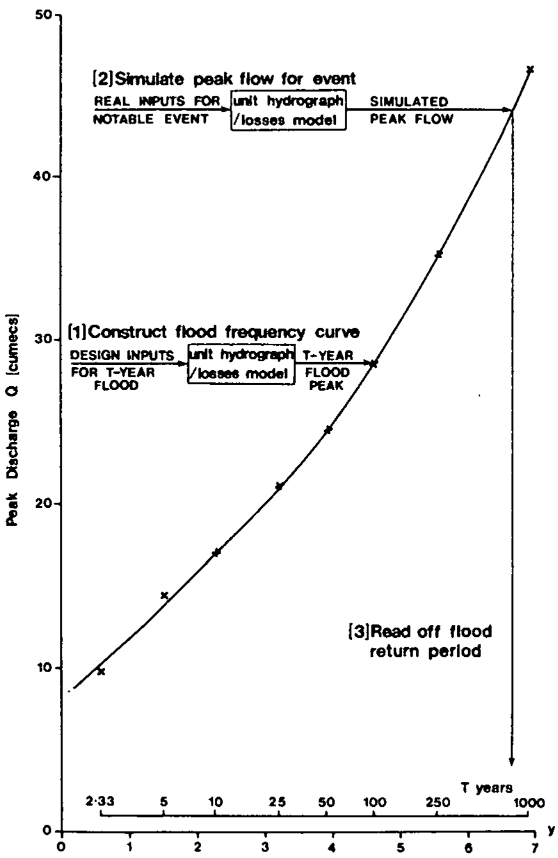


FIGURE 1 Stages in assessment of flood return period

3. DERIVATION OF INPUTS REPRESENTING THE NOTABLE EVENT

Rainfall inputs: depth, duration and profile

Specification of the rainfall inputs is ideally accomplished by deriving the catchment average hyetograph for the event. This is possible given a recording raingauge, and several daily raingauges on, or close to, the catchment. If more than one recording raingauge is available, it is simplest to represent the duration and profile of the storm using only the gauge most central to the catchment. Full use should be made, however, of daily raingauge data in estimating the catchment average storm depth. A certain amount of judgement has to be applied - for example, in deciding whether to divide a multi-burst storm into antecedent rainfall (contributing to the initial catchment wetness) and event rainfall (contributing directly to the flood).

If only daily raingauge data are available it may be necessary to rely on qualitative knowledge of the duration and profile of the storm. Local recollections, newspaper accounts, and Meteorological Office daily weather reports are possible sources of information. These can also be useful in corroborating the areal extent of the storm.

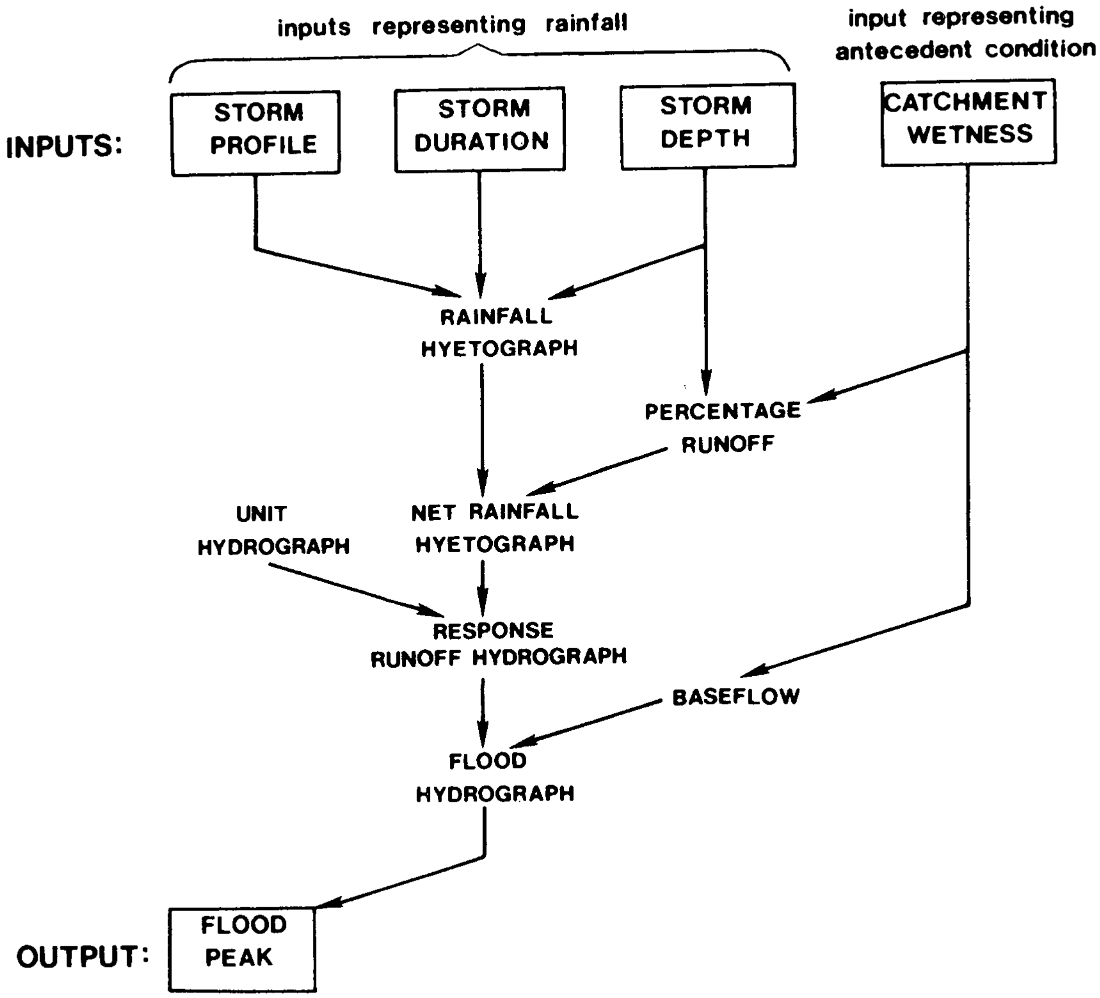


FIGURE 2 Influence of inputs on resultant flood peak for unit hydrograph/losses model

If little information can be found about the temporal distribution of rainfall it may be necessary to assume some theoretical storm profile. Adoption of a rectangular profile should be avoided unless evidence suggests that the rainfall intensity was almost constant. A better choice is to retain the 75% winter profile. This is broadly typical of flood-producing storms (see Vol I, Fig. 6.53) and allows the short cut to convolution to be used in Stage 2 as well as in Stage 1. Whether to adopt a peakier profile to represent a known thunderstorm is a matter of judgement. As regards storm duration, should it prove impossible to gain even a rough estimate, it is sensible to retain the design value calculated in Stage 1.

With the advent of routine archiving of radar-derived rainfall data at the Meteorological Office, an additional source of quantitative information should be available about future storms over much of England and parts of Wales.

Antecedent condition: the catchment wetness index

The catchment wetness index (CWI) is defined in terms of an antecedent precipitation index (API5) and the pre-event soil mositure deficit (SMD). In wet conditions in winter months, SMD can be assumed to be zero; otherwise, it is necessary to establish a representative value, perhaps by reference to the MORECS service operated by the Meteorological Office. Calculation of API5 requires raingauge readings for the five days preceding the storm. If the event rainfall began part way through a rainfall day, the API5 and SMD values should be adjusted by simple budgeting (see Vol I, Table 6.26).

4. EXAMPLE

An assessment is required of the return period of the 15 September 1968 flood on the River Bourne at Hadlow, Kent (catchment area 49.7 km^2).

Method	Example
Stage 1	
Apply standard rainfall/runoff method to synthesise flood peaks for a range of return periods, making use of short cut to convolution. (Steps 1-14 and Step 19 of Vol I, Section 6.8.2 and Supplementary Report No 9.)	Flood frequency relationship for Bourne at Hadlow by 'no data' rainfall/runoff method is shown in Figure 1.
Stage 2	
Derive inputs corresponding to notable event, i.e. determine or estimate:	For September 1968 event:
a Catchment Wetness Index at start of event (CWI = 125 + API5 - SMD);	*a* API5 at 09.00 on 14th: = 2.4 mm API5 at beginning of event: = 1.5 mm (NB decay factor of 0.5/day equivalent to 0.97/hr) SMD value (from met. station 19 km from catchment centroid): = 41.0 mm Hence CWI = 125 + 1.5 - 41.0 = 85.5
b Storm depth, P, averaged over catchment;	*b* P = 126.3 mm
c Storm duration, D;	*c* D = 16 hr
d Storm profile. Do not apply an areal reduction factor. If profile or duration unknown, assume standard design value (i.e. as in Stage 1).	*d* Profile: Figure 3 (Both from recording raingauge 8 km from catchment centroid)
Apply unit hydrograph/losses model to simulate resultant flood peak:	
e Calculate percentage runoff, PR;	*e* SPR = 27.5% (as in Stage 1) P = 126.3 mm CWI = 85.5 Hence PR = SPR + 0.22(CWI-125) + 0.1(P-10) = 30.4%
f Form net rainfall hyetograph; *g* Construct triangular unit hydrograph;	*f,g* The net rainfall and unit hydrograph are shown in Figure 3.
h Convolve net rainfall with unit hydrograph;	*h* Carrying out the convolution yields a peak response runoff: q = 43.4 cumecs
i Add baseflow allowance.	*i* Addition of baseflow allowance gives: Q = 44.1 cumecs
If the design storm profile - as opposed to a real profile - has been assumed, the short cut of Supplementary Report No 9 can again be used.	The simulated flood hydrograph is shown in Figure 3.
Stage 3	
The simulated flood peak is entered on the flood frequency curve (derived in Stage 1) and the corresponding return period read off.	From Figure 1, Q = 44.1 cumecs corresponds to a return period of about 750 years. Thus the flood of 15 September 1968 is assessed to be a very rare event for the Bourne catchment.

5. DISCUSSION

The simulation provides only a rough estimate of peak flow for the notable event. This could, and ideally should, be refined by analysing local flood data (see Supplementary Report No 13) and amending the parameters of the unit hydrograph/losses model accordingly. However, it is interesting to note that the assessment of return period is rather less sensitive to imperfections in the catchment model. This is because any slight bias in design use of the unit hydrograph/losses model (Stage 1 of the procedure) is likely to be compensated by a similar bias in simulating the notable event (Stage 2). An obvious example is if the standard percentage runoff (estimated from SOIL and URBAN) is in error. The consequent over- or under-estimation in design flood peaks will be mirrored by a similar over- or under-estimation in simulating the notable event, leaving the inferred return period much the same.

The method is probably most useful when designing works to cope with river levels experienced in a particular flood. If levels of inundation are well documented it is possible to use the assessed flood return period to provide a point on the damage frequency curve. To end on a more cautious note, the main weakness of the approach is that it accords much importance to conditions experienced in one event, which may or may not be typical.

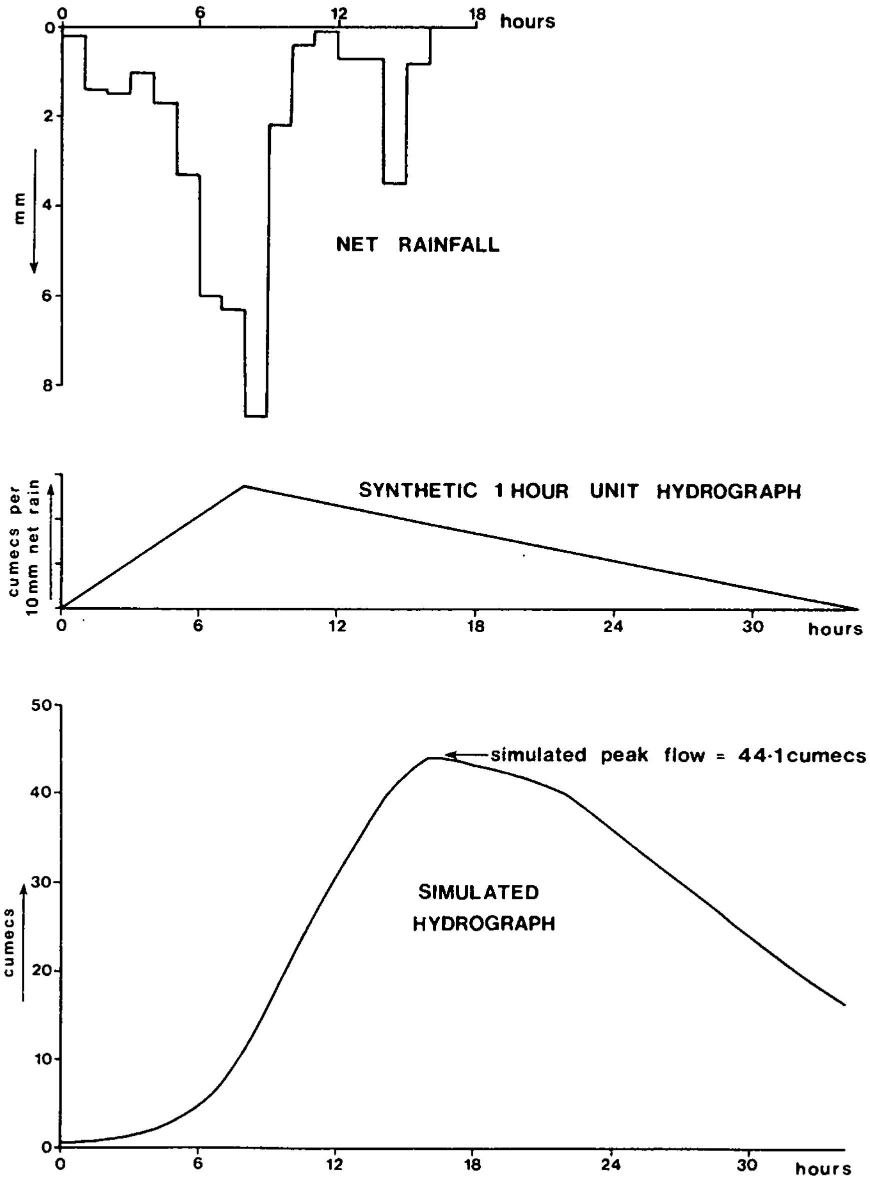


FIGURE 3 Net rainfall, unit hydrograph and simulated hydrograph for September 1968 event

Some suggestions for the use of local data in flood estimation

1. INTRODUCTION

The Flood Studies Report provides two methods of flood estimation at an ungauged site; a statistical approach and a rainfall/runoff approach. Both methods rely on regression equations to predict key parameters from catchment characteristics. The predictions are accompanied by relatively large 'errors of estimation' due to imperfections of the models.

In many applications, the uncertainty can be reduced by analysing flow records from nearby gauging stations or by acquiring a short flow or level record at the site of interest. This report consolidates advice given earlier, in the Flood Studies Report (NERC, 1975), the Manchester "Five Years On" conference proceedings (ICE, 1981) and elsewhere, on incorporating local data in flood estimation. Circumstances differ widely from one application to another, both in terms of the availability of data and the demands of the particular design problem. Thus it is only possible to outline general approaches to the use of local data, leaving the choice to the hydrologist to exercise personal judgement and experience.

This note considers the statistical and rainfall/runoff approaches separately (in sections 2 and 3) and, in section 4, tackles the problem of how to treat differing estimates from the two approaches.

2. THE STATISTICAL APPROACH

There are two stages to the statistical approach for estimating Q(T), the flood with return period T years at an ungauged site. Firstly, the mean annual flood ($\bar{Q}$) is estimated from catchment characteristics using a regional regression equation. Secondly, a regional growth curve is used to derive the multiplier $Q(T)/\bar{Q}$ for the required return period, T.

The treatment of local data is considered in two parts, in section 2.1 when there are records at the site of interest and in section 2.2 when some are available 'nearby'.

2.1 Data at the subject site

Estimation of $\bar{Q}$ With only a short record at the subject site there is a danger that estimates of the mean annual flood will be biased by freak occurrences. As a general rule it is suggested that estimates should be made from at least three years of data in order to improve significantly on estimates made from catchment characteristics. If up to 10 years of data are available the most suitable method is based on the POT series (I.2.7); with longer records, the annual maximum series should be used (I.2.3.)

Chapter 3 of the Flood Studies Report (FSR) details a number of methods for extending short records to improve the estimate of $\bar{Q}$ at a site. These include the use of adjacent longer records to extend a short record by regression (I.3.2), the extension of POT data (I.3.3) and the use of conceptual modelling driven by long rainfall records (I.3.4).

Q(T) estimation The FSR guide suggests that if there are more than 10 years of data - say N years - the record can be used to produce its own frequency curve but that it should be applied only up to T = 2N. For Q(T) where T > 5N the regional frequency curve should be used. For $2N < T < 5N$ a smooth curve should be drawn between the two points in such a way as to minimise discontinuities. Examples of transitions between frequency curves are given in Supplementary Report No. 14.

Section I.3.5 shows how Bayesian statistics can be used to weight different estimates of the distribution parameters of the flood frequency curves. In applying this technique the regional flood frequency distribution parameters are considered as prior information to be adjusted by parameters based on local data.

2.2 Data from nearby gauging stations

$\bar{Q}$ estimation Estimates of $\bar{Q}$ at the ungauged subject site, $\bar{Q}_{s,cc}$ may be improved by examining the local performance of the regional regression equation at sites where there are data (i.e. anywhere with 3 or more years of record as described in 2.1). The appropriate adjustment factor is a weighted combination of the ratios $\bar{Q}_{i,obs}/\bar{Q}_{i,cc}$ (see below for a definition of terms) derived for each of the local gauged catchments.

The catchments chosen should not only be 'nearby', e.g. no more than 50 km centroid-to-centroid separation, but should also be similar to the subject catchment. This means that the areas should vary by less than a factor of five and that the soil types, annual average rainfall, and general topography (slopes, stream frequency, extent of lakes) should be comparable. The technique is not suitable when either the subject site or nearby site is more than 20% urbanised; for these subject sites use Supplementary Report No. 5. Clearly, some catchments will be nearer, or more similar, or have longer lengths of record, than others and this can be reflected informally by adopting a weighting factor thus:

$$\bar{Q}_{s,adj} = \bar{Q}_{s,cc} \times \frac{\sum_{i=1}^{n} \frac{w_i \bar{Q}_{i,obs}}{\bar{Q}_{i,cc}}}{\sum_{i=1}^{n} w_i} \qquad (1)$$

where $\bar{Q}_{s,adj}$ is the adjusted value of $\bar{Q}$ at the subject site

$\bar{Q}_{s,cc}$ is the value of $\bar{Q}$ at the subject site estimated from catchment characteristics

$\bar{Q}_{i,obs}$ is the value of $\bar{Q}$ calculated from the flow record at the ith nearby station

$\bar{Q}_{i,cc}$ is the value of $\bar{Q}$ at the ith nearby station estimated from catchment characteristics

n is the number of nearby stations

w_i is a weight (or score) indicating the relevance of the ith nearby station

It should be noted that all the estimates of $\bar{Q}$ from catchment characteristics should be made using the same prediction equation. When there are two or more regions involved, therefore, the regional coefficient should either be replaced by the national coefficient or omitted altogether. If one of them is the 'Thames, Lee and Essex' region, it likewise follows that the national, six-variable equation should be used in preference to the special equation developed in the FSR for that region alone.

If it is desired to give weight to the $\bar{Q}_{s,cc}$ estimate itself this is achieved by including the subject site as one of the 'nearby stations', setting its ratio of observed to predicted $\bar{Q}$ values to unity, and giving it a suitably large weight. Assignment of weighting factors may be aided by a map showing the ratios of observed to predicted $\bar{Q}$ values. (Although there are serious reservations about extending this method to the full flood frequency curve, an excellent example of how it can be applied to the mean annual flood is given by Archer (1981)).

A special case of this approach occurs when there are a number of subject sites all on the same river which also has one or more gauges. Appropriate weighting factors can then be derived by the rules for addition of errors and are determined by the relative positions of the subject site, each gauged site, and any confluence with a major tributary. The Institute of Hydrology has produced some graphs to help choose these weighting factors. Please direct enquiries to M A Beran.

It has been found particularly helpful in this instance to plot a graph of $\bar{Q}$ against distance from the source. Figure 3 of Beran (1980) shows such a diagram for the Yorkshire Rother which was obtained by repeating the calculations of $\bar{Q}_{s,cc}$ down the length of the river. One may also plot the values $\bar{Q}_{i,obs}$ on such a diagram together with standard errors of estimates of both procedures to help judge the weighting factors. The overall impression of how the flood augments and decays along the river is itself a valuable extra source of information that can be checked against field observation.

Q(T) estimation Like the regional no-data equations the region 'growth curves' represent the average behaviour of catchments within that particular region. The departures from this average which undoubtedly occur are very difficult to quantify with typical record lengths. Also our present understanding of the causes of such departures is limited to speculations that (1) rainfall 'growth' curves, (2) antecedent wetness, (3) speed of response and (4) soil type, would play a part.

Given our inability to quantify these controls we must adopt a cautious approach to the use of local data to adjust the FSR regional growth curves. Where a 'nearby' station with say 25 or more years of record is only a few kilometres away from and on the same river as the subject site then the gauged growth curve (with the obvious allowance for differences in $\bar{Q}$) would clearly be applicable to the subject site. However, the more general recommendation is that locally derived growth curves should only be pooled if a large number of station years of data and expertise in flood statistics are available. Also the pooling of data should be restricted to catchments in a reasonably homogeneous region as suggested by the controlling factors listed above. With these restrictions in mind, the most usual outcome will be that a locally refined $\bar{Q}$ estimate is used in conjunction with the FSR 'growth' curve of the region containing the subject site.

3. THE RAINFALL/RUNOFF APPROACH

The rainfall/runoff approach to flood estimation has three main components each of which is, in theory, open to adjustment in the light of local data:

- the rainfall statistics package
- the unit hydrograph/losses model
- the design storm (to preserve the link between rainfall and flood peak return periods)

However, for reasons which follow, it is firmly recommended that adjustments are confined to elements of the unit hydrograph/losses model and not to rainfall statistics or design storm construction.

The rainfall statistics package (FSR Vol II) is, like any other analysis of data, subject to revision with regard to both the numerical values it produces and the basic methodology. It is felt, however, that such revision should only be contemplated when the entire national data set is reworked. Looking to the future, the better spatial information to be derived from the radar network could well lead to a fundamental change in the way we express rainfall statistics. For the present, the recommendations must remain that local rainfall data should not be used to revise the statistics without the explicit approval of the Meteorological Office. One of the most challenging studies in this regard was made by Bootman and Willis working with rain data from the Somerset area. Although first raised at the Birmingham seminar (Supplementary Report No. 3, para 6) more details were given in discussion at the Conference on "The Flood Studies Report - Five Years On" in Manchester (Proceedings, pp 62-68). However, refer also to D Warrilow's response which dispels any feelings of more global uncertainties.

The unit hydrograph/losses model is the core of the rainfall/runoff approach, and adjustments based on local data may be made either to the unit hydrograph shape or to the equation which predicts the percentage runoff. Sections 3.1 and 3.2 consider the two cases: firstly when there are data at the site of interest and secondly when data are available nearby.

In the choice of design storm, the analyses which necessarily precede any recommendations are so extensive that there is little chance of any user justifying a change in the existing procedure. The recommendation remains, therefore, to use the design storm construction described in the FSR (for rural catchments) and that described in Supplementary Report No 5 (for urbanised catchments).

3.1 Data at the subject site

Use of an observed unit hydrograph If a section is reliably rated, and suitable hourly rainfall records are available, methods outlined in the FSR (I.6.4) may be used to derive unit hydrographs. An average 'observed' unit hydrograph (Boorman & Reed, 1981) can then be used to provide an improved estimate of Tp or, alternatively, to replace the synthetic triangular unit hydrograph. In the latter case the design procedure strictly requires a unit hydrograph in the three-parameter form used in the FSR analysis. However, if this is considered unduly restrictive in a particular case, a general curvilinear unit hydrograph may be incorporated. It is recommended that adjustments to synthetic unit hydrographs should normally be based on analysis of at least five recorded flood events. However, this criterion might be relaxed if the hydrologist is particularly confident about the suitability of the available data, namely: the relative timing of the rainfall and flow data, the uniformity (or at least typical pattern) of the spatial rainfall distribution, and the lack of any markedly unusual circumstances (e.g. very dry antecedent conditions, lying snow).

The use of locally observed 'lag' times between the centroid of the causative rainfall pattern and the peak of the resulting flow (or stage) hydrograph to improve estimates of Tp (namely Tp = 0.9 LAG) is described in the FSR (I.6.5.3).

Use of observed percentage runoff data At the ungauged site the percentage runoff (PR) is calculated using the following equation (I.6.5.8):

$$PR = \underbrace{95.5\ SOIL + 12.0\ URBAN}_{\text{fixed terms (SPR)}} + \underbrace{0.22\ (CWI - 125) + 0.1\ (P-10)}_{\text{dynamic terms}} \quad (2)$$

The first two terms of this equation are fixed by soil and land use, do not vary from event to event, and define the 'standard' percentage runoff (SPR); the second two terms reflect the dynamic behaviour of a catchment where the percentage runoff varies with the antecedent wetness of the catchment and magnitude of the storm event. The errors associated with the PR equation are such as to make it unlikely that there would be sufficient local data to justify a change in the dynamic terms. However, the standard percentage runoff accounts for much of the difference between catchments and this can be refined by using data from the site of interest. With regard to rainfall data, the emphasis here is more on the adequate definition of the total event rainfall over the catchment, with a reasonable indication of its start and end times, rather than an hourly distribution. The chosen events (at least five) should be as large as possible to minimise the effect of the hydrograph separation method on the calculated volume of runoff.

The observed SPR for each event is found by reversing the above equation to yield

$$SPR_{obs} = PR_{obs} - 0.22\ (CWI - 125) - 0.1\ (P-10) \quad (3)$$

The various event values are averaged to give the best estimate of SPR at the site.

Average observed values of SPR for 175 catchments are available from the Institute of Hydrology.

Estimating SPR from gauged daily flow data In the absence of suitable rainfall data, a sequence of average daily flows can lead to a better estimate of SPR than may be obtained from catchment characteristics. Figures 1 and 2 show how separation rules may be used to define a base flow separation line (NERC, 1980). A base flow index, BFI, is then calculated as the ratio of separated to total runoff. The index was devised initially for a study of low flows (NERC, 1980) but has been shown to be valuable also for flood estimation. This is because (1 - BFI) is a measure of the quick response proportion of the hydrograph and therefore relates to flood characteristics such as SPR.

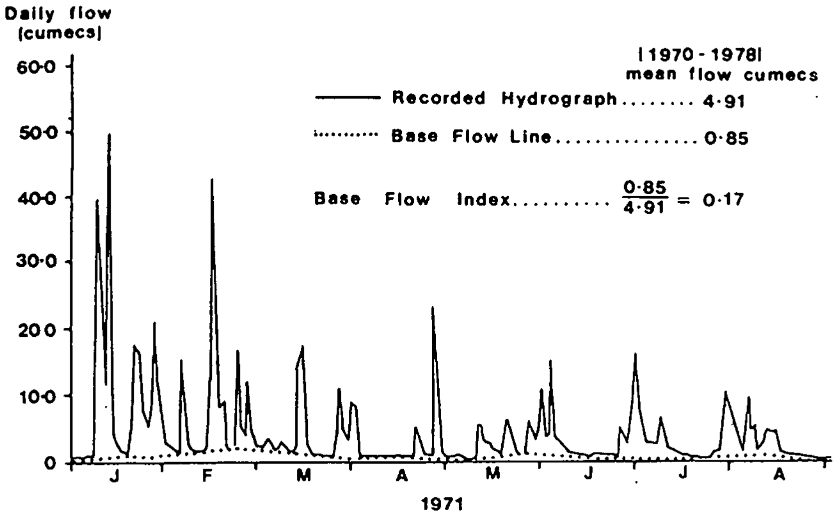


FIGURE 1

Base flow separation for an impermeable catchment - the Falloch at Glen Falloch

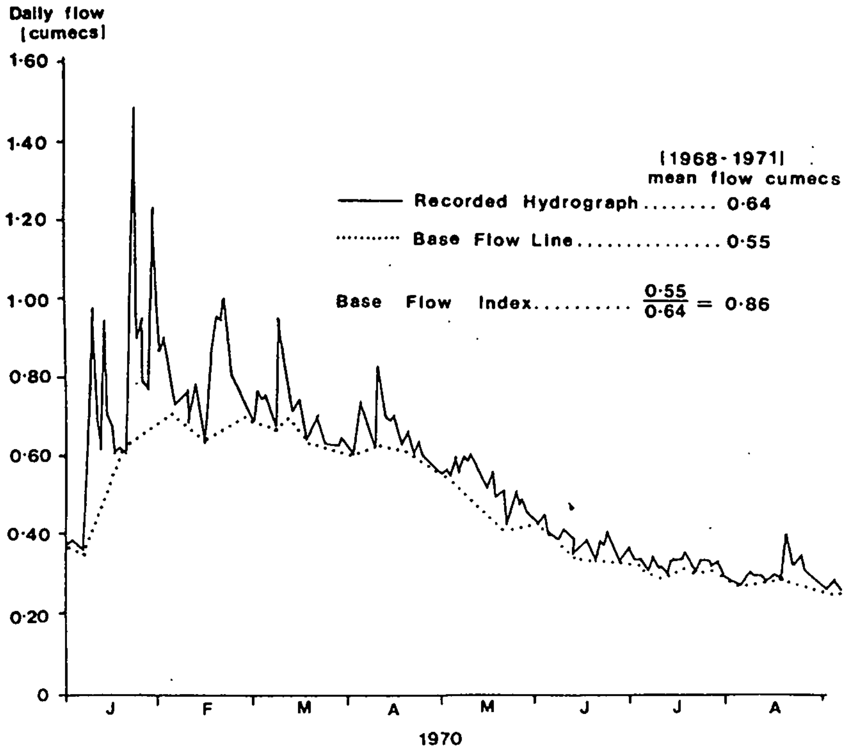


FIGURE 2

Base flow separation for a permeable catchment - the Pang at Pangbourne

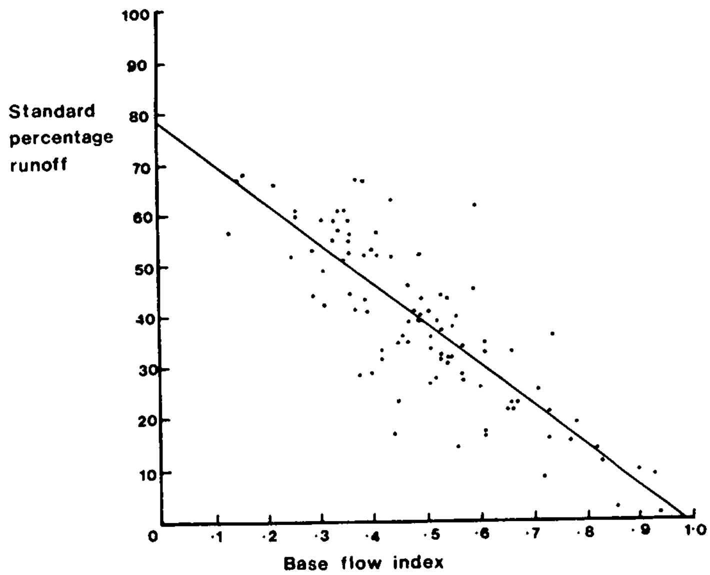


FIGURE 3

Relationship between observed standard percentage runoff and base flow index

Figure 3 shows the relationship developed between SPR and BFI from analysis of flood event and daily flow data for 104 catchments in the UK. The regression equation is:

$$SPR = 78 - 79.2\ BFI \qquad se = 9.01 \qquad r^2 = 0.69 \tag{4}$$

predicting SPR = 0 when BFI = 0.98. This behaviour in the extreme is confirmed by catchments where the maximum observed BFI is 0.98, which only occurs on drift free permeable chalk catchments having virtually no quick response runoff. Conversely the lowest observed BFI of 0.17 predicting SPR of 65% accords well with observed maximum SPR values. Equation (4) may be compared with the following equation based on the same 104 catchments which relates SPR to SOIL from the WRAP map.†

$$SPR = 122.1\ SOIL - 7.6 \qquad se = 11.54 \qquad r^2 = 0.50 \tag{5}$$

The lower r^2 value and higher standard error suggest that estimation of SPR from BFI is to be preferred to estimation from catchment characteristics when BFI is itself estimated from flow records. The approach is thus suitable when a daily flow record is available at a site of interest but where it is not possible to compute SPR from flood event data as described above. This would include situations where the gauging station is unsuitable for measuring high flood discharges or where insufficient events are available for analysis.

BFI values for about 1100 gauging stations in the UK are available from the Institute of Hydrology.

3.2 Data from nearby gauging stations

Adjusting predicted unit hydrograph shape If time to peak (Tp) information is available near to rather than at the design site then Tp at the site of interest may be adjusted thus:

$$Tp_{s,adj} = Tp_{s,cc} \times \frac{Tp_{g,obs}}{Tp_{g,cc}} \tag{6}$$

(This matches the notation used earlier in that suffixes s,g refer to subject and gauge sites respectively; cc means estimated from catchment characteristics.)

An 'observed' unit hydrograph may similarly be transferred by multiplying each ordinate by $Tp_{g,cc}/Tp_{s,cc}$ and each abscissa by $Tp_{s,cc}/Tp_{g,cc}$.

It is recommended that these adjustments should only be made if

(a) the catchments involved are of the same order of magnitude say, the larger being no more than five times the smaller and

(b) the gauged site is either upstream or downstream of the subject site.

This advice not to look beyond the subject site's own river system is based on the belief that estimation errors in Tp prediction are due primarily to unmeasured properties of the stream network and local topography rather than to any consistent, but unexplained, regional effects. It is, therefore, likely that there will be only one gauged site to be used for Tp adjustment. But if there are two or more - then equation (6) can be extended to incorporate a weighted average of the ratios, of observed to predicted Tp values, in the same way as with $\bar{Q}$ (equation (1)).

† *The Winter Rain Acceptance Potential map or 'Soil' map revised 1978 - see Supplementary Report No. 7. Further minor corrections to the map were made in 1981 and are included in the version in Vol. V of the FSR second binding, available from 1982 onwards.*

Adjusting percentage runoff estimates If standard percentage runoff (SPR) can be 'observed' as outlined above on nearby catchments which are broadly similar (geology, topography, land use) to the subject site, the information can be used to adjust, or completely replace, the predicted SPR value. The method is rather more subjective than was suggested for $\bar{Q}$ and Tp but it is felt that the link between percentage runoff and soils/geology is more clearly causal and therefore more amenable to local adjustment.

Values of observed and predicted SPR should be plotted on a catchment boundary overlay map and superimposed on the WRAP map, and, if available, a solid and drift geology map. Any tendency for observed SPRs to be consistently greater or less than indicated by the WRAP classification can then be applied to adjust the predicted SPR at the site of interest. This can be carried out with more confidence if the implied error in the WRAP map can be related to local soil associations or catchment geology, or to known difficulties in assigning a given soil type to a particular WRAP class. Liaison with the regional soil survey and field visits to the gauged and ungauged catchments may assist in the interpretation of any SPR anomalies. Reference to Farquharson et al. (1978) may also be found helpful.

The method based on BFI can also be extended. Methods have been developed (NERC, 1980) for estimating BFI on ungauged catchments from catchment geology. With a greater density of observed BFI than SPR values there is a good chance that a nearby catchment similar to the catchment of interest will have a BFI value which could be directly transferred or perhaps modified slightly and then used in Equation (4). The estimation accuracy will be much nearer to that of Equation (5) than if a directly measured BFI value is available. However, the ability to estimate SPR outside the range 15-50% and the greater local detail often available from large scale geology maps are valuable advantages.

4. RECONCILING ESTIMATES USING DIFFERENT APPROACHES

The majority of catchments used in the FSR to calibrate the rainfall/runoff approach were also used in the $\bar{Q}$ regression approach and the design storm necessary to the former was constructed so that the synthetic frequency curves resulting from its use would, on average, match observed curves. On the other hand, the two approaches use different types of data and have different applications.

Although the original intention had been that the rainfall/runoff approach would be confined to maximum flood estimation (i.e. beyond the range of flood statistics), the FSR in fact allows its use for any size flood not only because it may be needed to give the complete hydrograph shape but also because it is recognised that hydrologists will wish - on some occasions - to try more than one technique. Inevitably, therefore, the two approaches are compared and sometimes found to produce very different estimates. The hydrologist will not be surprised at this but would look to local data to help reduce the differences. In the experience of IH users of the FSR, such differences do reduce when local data are used as described in Sections 2 and 3 above. A word of caution: the two methods might agree very well when applied in the 'no-data' mode but they might both be 'wrong' - local data should <u>always</u> be used if errors are to be minimised. Figure A2(b) on p. 21 of FSR I 'quantifies' this caveat. It shows that if a catchment appears from its catchment characteristics to be more flood prone, in terms of SPR, than is actually the case then a similar bias is very probable with the $\bar{Q}$ equation.

Whether local data are used or not, there will be some differences remaining between the two approaches. It would be unwise to give hard and fast rules for their reconciliation. Comparisons carried out at IH suggest that greater weight should be given to the $\bar{Q}$ approach when T is small but because of the greater uncertainty in regional flood frequency curves we attach more reliance to the rainfall/runoff approach when T is large.

This recommendation presumes that either local data have not been used at all or they have been applied with equal skill to both approaches. In practice, the most likely case is that local data will be more applicable in one approach than the other and the relative weightings would be affected accordingly.

Finally, unless one is concerned only with preliminary estimates or with very minor works, flood estimation remains a task for the experienced hydrologist. Strategic research may yield national design criteria but their proper interpretation and application require expert judgement. That requirement is increased, rather than decreased, by the availability of local data.

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Flood Studies Supplementary Report No 14 August 1983

Review of regional growth curves

1. INTRODUCTION

The 'regional growth curves' of the Flood Studies Report provide, for the majority of practical circumstances, the recommended procedure for estimating the T year return period flood Q(T) from the mean annual flood, $\bar{Q}$. The region curves have been the subject of many enquiries from users concerned about the causes of the apparent regional differences, the significance of those differences, and whether any modification can be justified for catchments close to a region boundary.

Due to time limitations prior to publication of the FSR there was no opportunity to test thoroughly whether the apparent differences between regions were real. It was considered that given the large variability of individual station growth curves, regionally pooled curves must be a more precise way of extrapolating to rare return periods. This report summarises recent findings which examine whether the differences in growth curves between regions are statistically significant and presents new growth curves for estimating floods with return periods greater than 100 years. The reader is referred to Stevens and Lynn (1978)† for more detailed information.

2. STATISTICAL SIGNIFICANCE OF REGION GROWTH CURVES

An analysis of variance was used to test whether the variation in growth curves within a region is large by comparison with differences between regions. In this test a given regional growth curve is regarded as the average of the individual station growth curves, which in turn depart from this average. The test examines whether these departures are large by comparison with the difference between region growth curves.

The test makes use of all the individual $Q/\bar{Q}$ values within a region which lie in a given range of reduced variate, y.

Consider the $Q/\bar{Q}$ values for stations in two regions A and B. Clearly some numerical difference will be found between the average $Q/\bar{Q}$ of each region $\bar{x}A$ and $\bar{x}B$. However, the degree to which this numerical difference can be said to be statistically significant - i.e. not just due to random sampling - depends on the variability of the individual $Q/\bar{Q}$ values around their own averages. An analysis of variance was used to test for differences between the means and this is described on pages 6-13 of Stevens and Lynn. A large number of comparisons of regions, taken in pairs, triples, etc. and using different y ranges from -1.5 to -1.0 up to +3.5 to +4.0 (up to about 50 year return period) were made.

In one test we examined the possibility that all 10 UK regions (FSR I Table 2.1) have the same region curve. This was firmly disproved (except in y interval 3.0 to 3.5) thus justifying the need for some regionalisation. In fact, few large groups supported the

† Stevens, M.J. and Lynn, P.P. 1978. Regional growth curves. IH Report No. 52.

hypothesis of shared regional curves. Region 2 does not exhibit much similarity with any other region or group. A group made up of regions 4, 5, 6, 7 and 8 (south and east Britain) showed increasingly similar behaviour with increasing return periods.

3. DISTRIBUTION FREE TESTS

A second test examined the possibility that the use of a particular probability paper may have influenced the impressions about separation of regions.

The Chi-square test was used to determine whether the frequency histograms defined by the $\frac{Q}{\bar{Q}}$ values from each region or groups of regions came from the same (unspecified) distribution. A second test, the Kolmogorov-Smirnov test, was applied to the same pairs and groupings as the Chi-square test and supported it in every case.

The results of these tests are described on pages 13-17 of Stevens and Lynn and are seen to be broadly similar to the ANOVA tests of Section 2. Some pairs from regions 1, 3, 9 and 10 can be grouped and various combinations of regions 4, 5, 6, 7 and 8 were not distinguished showing again that, whilst not identical, the growth curves are indeed quite similar. Region 2 again appears to be different to all others.

4. TESTS ON HIGH FLOODS

The tests described so far have been weighted towards low and moderate return period floods because the vast majority of the data points are from these frequencies. Values of $Q/\bar{Q}$ greater than 2.0 were often considered as a single interval. Thus this set of large floods was examined separately. The number of such events ranges from six in the data of regions 2 and 10, up to 21 in region 6. The median test described on page 17 of Stevens and Lynn tests the hypothesis that the high floods from any two or more regions under test share the same overall median. Two major groups 4, 5, 6, 7, 8 and 1, 2, 3, 9 and 10 were tested and similarities 'within' and differences 'between' were noted. Unfortunately the test was not very discriminating and did not prove or disprove any new conjectures.

5. PRACTICAL RECOMMENDATIONS

The primary finding is that there are not strong enough grounds for concluding that any two regions are identical. Thus at lower return periods we advocate continued use of the FSR regional growth curves.

Since it has been shown that it is not possible to use one curve for the whole country, serious error may be introduced by, for example, using the GB curve for region 5 for $T > 500$ years. However, the similarities between regions could be very useful in pooling the data to obtain curves valid at higher return periods. A more realistic method is to pool the data from regions 4, 5, 6, 7 and 8 to obtain a curve for use at higher return periods for that major region. Similarly, a pooled curve can be derived for regions 1, 3, 9 and 10. Region 2 remains a problem. Since both geographically and in the appearance of its growth curve it is closer to 1, 3, 9 and 10 than to the other regions, it seemed sensible to pool it with these regions. Thus two pooled curves are obtained, one for NW Britain and one for the SE. The two curves are shown in Figure 1.

One of the difficulties of using two different growth curves (e.g. regional curve up to $T = 100$ years then the NW curve for higher return periods) to encompass a range of return periods is the discontinuity which arises when switching from one to the other. Current recommendations applying to the use of the GB curve include drawing an eye-guided smooth curve to bridge between the curves. A new recommendation which reduces this discontinuity

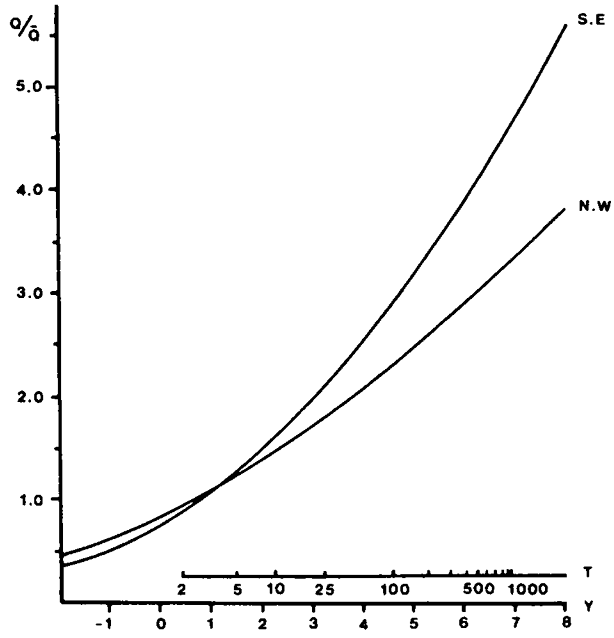

FIGURE 1

The pooled growth curves obtained by treating north-west and south-east Britain separately

is to apply values of $\frac{Q(T)}{Q(100)}$ from the new NW or SE curves to the original FSR curve ordinates at T = 100. The resulting new ordinates are shown in Table 1 and the curves are drawn on Figure 2 where the original FSR curves are plotted up to T = 100 but are then drawn through the new ordinates at T = 500 and T = 1000.

The discontinuity has been removed but a small contra-flexure is visible on some curves when drawn on Gumbel paper. This revision changes the recommended procedure for flood estimates in excess of T = 100; for lower return periods the growth factors remain unchanged. The Irish data set was not included in this study and so the Irish curve remains unchanged. The Great Britain curve is superseded.

TABLE 1 REGION CURVE GROWTH FACTORS

Region		Hydrometric areas	Return period: 2	5	10	25	50	100	500	1000
NW	1	1-16,88-97,104-108	0.90	1.20	1.45	1.81	2.12	2.48	3.25	3.63
	2	17-21,77-87	0.91	1.11	1.42	1.81	2.17	2.63	3.45	3.85
	3	22-27	0.94	1.25	1.45	1.70	1.90	2.08	2.73	3.04
	9	55-67,102	0.93	1.21	1.42	1.71	1.94	2.18	2.86	3.19
	10	68-76	0.93	1.19	1.38	1.64	1.85	2.08	2.73	3.04
SE	4	28,54	0.89	1.23	1.49	1.87	2.20	2.57	3.62	4.16
	5	29-35	0.89	1.29	1.65	2.25	2.83	3.56	5.02	5.76
	6/7	36-44,101	0.88	1.28	1.62	2.14	2.62	3.19	4.49	5.16
	8	45-53	0.88	1.23	1.49	1.84	2.12	2.42	3.41	3.91
Ireland			0.95	1.20	1.37	1.60	1.77	1.96	2.40	2.60

For return periods higher than 1000 years it is recommended that the Unit Hydrograph Losses model be used for flood estimation. However, for comparative purposes more extreme growth factors for the two new pooled curves may be calculated from:

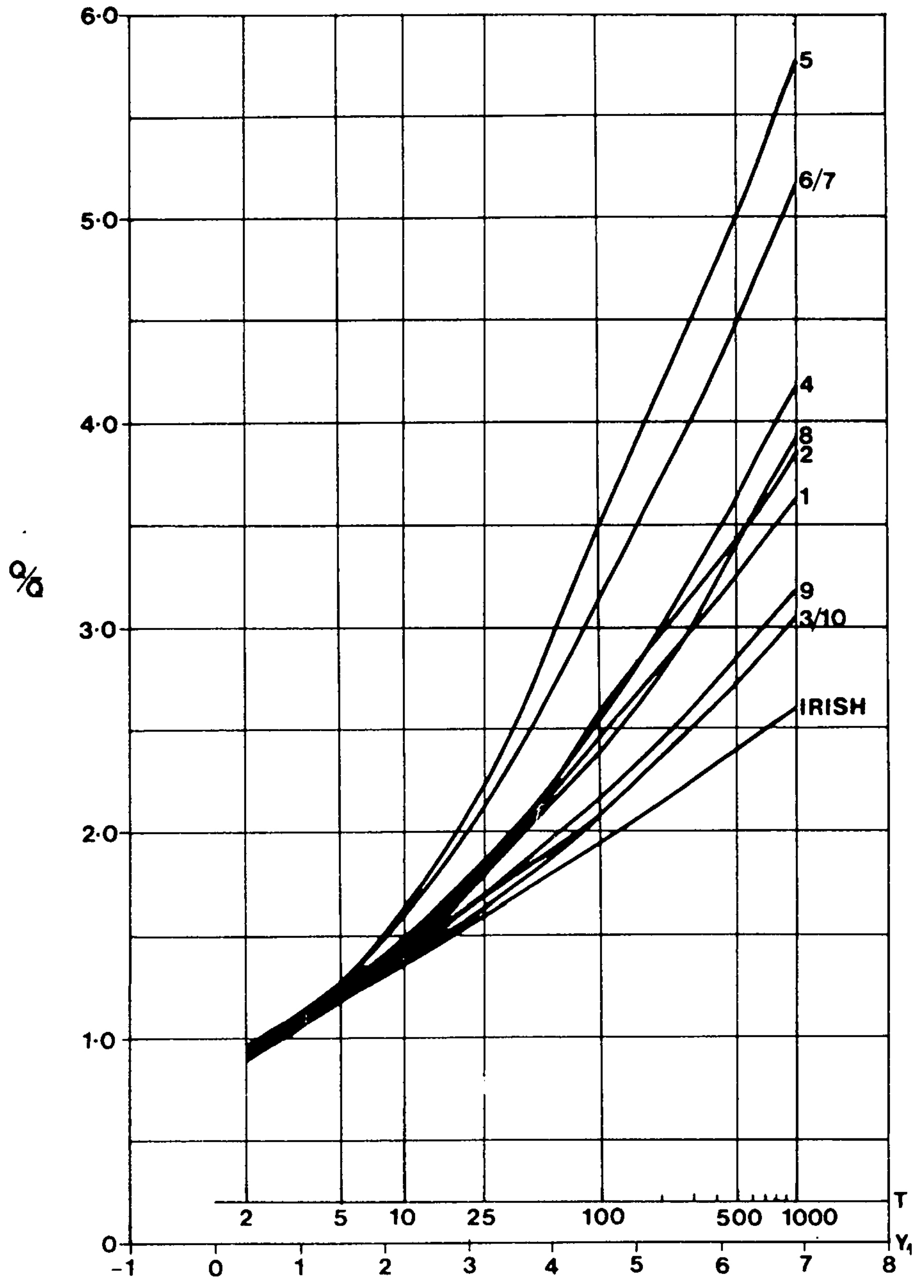

FIGURE 2 New regional growth curves (Revised for T > 100, extended to T = 1000)

$$\frac{Q}{\bar{Q}} = u + \alpha(1 - e^{-ky})/k$$

using the following coefficients

	u	α	k
North West	0.85	0.222	-0.12
South East	0.73	0.320	-0.15

It is recommended that the ratio of growth factors calculated at Q(T) and Q(100) from the appropriate (North West or South East) coefficients is applied to the regional growth factor at T = 100 shown in Table 1. See FSR page I.42 for the relationship between return period T and reduced variate y.

6. CATCHMENTS CLOSE TO REGION BOUNDARIES

Some users have suggested that the average of two region curves should be used for catchments close to a region boundary. However, given our inability to predict growth curve shape from catchment characteristics and the lack of evidence for a smooth trend in growth curve shape across regions the current recommendation must remain that the average curve for a region be used everywhere within that region. Current research at IH is aimed at providing a more rational basis for pooling individual station growth curves by identifying those factors which are responsible for differences in flood frequency behaviour between catchments. This problem is discussed in the context of the use of local data to adjust region growth curves in Supplementary Report No. 13, Section 2.2.

7. CONCLUSIONS

Statistical tests of the FSR region growth curves have justified their continued use as representing the average flood frequency behaviour for catchments within the stated geographical regions. Departures from this average undoubtedly occur although a scientific explanation for such departures is still lacking. Some practical difficulties of applying regional and national curves have been solved by forming two 'sub-national' groups for the north-and-west and for the south-and-east.

The revisions are presented in the form of regional growth curves amended for $T > 100$ and extended to $T = 1000$. For $T > 1000$ it is recommended that the Unit Hydrograph Losses model is used for flood estimation.

FSR NEWSLETTER August 1983

WHY A NEWSLETTER ?

Since publication in 1975 several developments of the Flood Studies Report have enhanced its value to users, and even now the Institute of Hydrology is carrying out a number of projects designed to improve the confidence with which the methods may be applied. In parallel with work at the Institute some users are also investigating ways of overcoming apparent anomalies within their own regions. Because of these disparate investigations it has been suggested that users of the FSR and of other guides might find it helpful to be informed of current developments and progress.

This newsletter summarises work recently completed or in progress but it is concerned mostly with investigations at the Institute. In order to widen the scope of a possible sequel, readers having information which they feel would be useful in an up-to-date review are invited to send brief details to Mike Lowing at the Institute.

Topics covered in this newsletter include:

- the survey of (mostly Institute) research activities in flood hydrology
- the recent transfer of national surface water archiving duties to Wallingford
- digital maps in hydrology
- flood studies in Europe and worldwide
- case studies of FSR applications - a request for contributions
- a summary of recent design guides.
- the proposed British Hydrological Society.

MOVING ON FROM THE FSR

At Birmingham in March 1977 (Supplementary Report No 3) and again at Manchester in July 1980 (the ICE Conference 'FSR - Five years on') users took the opportunity to debate FSR deficiencies. Following the Manchester conference, Phil Johnson (Chairman of that conference's organising committee) penned a forward look* with a list of 18 further R & D tasks (Table 1 herein) that were needed if the accuracy of flood estimation was to be improved. Some of the tasks, which are referenced from the following notes, are actively under research at the Institute of Hydrology (IH) and elsewhere but there are several topics which remain to be studied.

Regions - Task 1

A task which, at IH, has various degrees of support among different researchers. An investigation of contouring $\bar{Q}/AREA^n$ is planned, for catchments which are lightly affected by 'urban' and 'lake' factors in the intuitive belief that the other key characteristics are all, to some degree, in common descent from an 'uplandishness' factor. Another aspect of the problem is the variation, from place to place, in flood season (the time of year when floods occur most commonly). A project on this topic is well advanced and has included study of the combination of circumstances which cause floods in different regions and catchment types. There are grounds for optimism that, in one way or another, it will eventually be possible to achieve a predicted $\bar{Q}$ which varies smoothly with location or some other recognizable factor.

**Proc. Instn. Civ. Engnrs. Pt 1*, 1981, 70 (Nov), 833-843.

TABLE 1 Priority tasks to improve and extend *Flood studies report*

1. Develop alternatives to, or better definitions of, regional boundaries and determine, if contouring methods can be used to represent gradations of flood parameter values between regions, whether different treatment of flood data may reduce the appearance of regional grouping.
2. In part support of task 1, study the variable $Q/\bar{Q}$ and
   (*a*) validate the assumption that regionalized curves of frequency distribution do not lead to serious errors of estimation;
   (*b*) determine to what extent values of $Q/\bar{Q}$ at different gauging stations are independent to validate data pooling for regionalized analyses of frequency;
   (*c*) investigate how growth curves of $Q/\bar{Q}$ frequency are influenced and biased by the use and estimate of $\bar{Q}$;
   (*d*) determine most appropriate transformation of $Q/\bar{Q}$ to avoid bias in correlation of magnitude with frequency;
   (*e*) develop and provide tables of values for unbiased plotting positions in frequency distributions not given in the *FSR*.
3. Determine and give more precise guidance on effects of error in abstracting data from maps on accuracy of flood estimation.
4. Develop a programme, including specially instrumented catchments, to measure wider ranges of conditions affecting flood discharge, and to enable study of catchments having particular characteristics not adequately covered by original investigations (e.g., catchments with soil type 1, 2 or 3).
5. Consider to what extent and how task 4 can be implemented and rationalized nationally to support further work on and improve estimation for urban catchments; reservoired catchments; the influence of field and land drainage; and the effects of afforestation.
6. Study further, with a more extensive data base, the sensitivity of flood estimation to parameter values, especially SOIL (and its classification) and CWT and how this improves with better information on SMD.
7. Make more serious effort to gather information on the effects of frozen ground on river discharge, especially its frequency.
8. Examine the adequacy of methods for catchments less than 20 km^2, especially very small catchments having areas less than, say, 1 km^2.
9. Further study and develop methods using non-hydrological data to substantiate and extend hydrological estimation of floods and their trends and cycles in time.
10. Establish better methods to calculate variations of flood risk within the period of a year.
11. Prepare user guidelines for the bivariate frequency analysis of river floods and tidal levels.
12. Assess the degree of interdependence between data from different raingauges in a region and develop methods and information allowing for this phenomenon in the frequency analysis of regionally pooled data.
13. Improve accuracy of estimating rainfall for durations less than one day especially for urban drainage design.
14. Seek more information through radar hydrometerology of point and area rainfall at higher elevations (i.e., altitudes over 500 m AOD) for which raingauge data hardly exist.
15. With the help of task 14 study properties of actual storms to improve estimation of statistical averages for area reduction factors and storm intensity profiles.
16. Develop special study to obtain better assessment of extreme storms for PMP estimation, allowing for
   (*a*) contemporaneous maxima of storm efficiency, humidity, and temperature in winter and summer;
   (*b*) development of realistic storm profiles and their variations;
   (*c*) possible use and definition of a design maximum precipitation (DMP) as an alternative to PMP;
   (*d*) why, on the assumption that PMP is a physically realistic concept, upper bounded frequency distributions may not seem to fit data (including to what extent raingauge catch deficiency is a contributory cause).
17. Produce information, mapped or otherwise presented, indicating differences between PMP values and *FSR* estimates of rainfall corresponding to a return period of 1 in 10,000 years.
18. Carry out a new programme of research on snow to replace or modify existing recommendations:
   (*a*) to determine and recommend for implementation, simple, robust and sufficiently accurate methods of measuring snow over an area;
   (*b*) to review and ensure that methods of estimating amounts and frequency of water equivalents are correct;
   (*c*) to establish with greater certainty effective intensity of runoff rates from snowmelt;
   (*d*) to assess the joint probability of water equivalents in snow packs, high and sustained runoff rates in snowmelt, and winter rainfall intensities and durations, and frozen ground conditions;
   (*e*) to establish a method, or methods, which will suitably simulate the combined occurrence of phenomena in (*d*).

Pending these developments, there is a particular region which seems poorly served by existing recommendations. Flood data and results are just coming onstream for Northern Ireland. While these have not been formalized yet into firm recommendations, a preliminary scutiny of the data seems to confirm the long held belief of local hydrologists that the 'Irish' $\bar{Q}$ equation does indeed underestimate flood magnitude in the north. The Department of Environment's Civil Engineering Division at Hydebank, 4 Hospital Road, Belfast, will give guidance now and it is hoped to produce a Supplementary Report on the subject in due course.

The plotting and pooling of flood frequency curves - Task 2

Studies of inter-station dependence continue at IH and a method for adjusting the plotting position of the normal distribution has been derived. Indications are that the correlations are too weak to invalidate the station-year assumptions made when pooling flood frequency data. However, further work on more 'subtle' correlation structures is underway at IH and at the Dutch Royal Meteorological Institute. The statistical problems here are prodigious and were discussed at a joint seminar in April 1983 between professional statisticians and environmental scientists, including hdyrologists.

Unbiassed plotting positions and formulae can be produced for distributions other than those given in the FSR; IH would respond to any such request as it would for further details of its findings concerning the effects of dependence.

Further progress under these and other headings of Task 2 depends partly on the development of new theory on derived frequency distributions and partly on the acquisition of longer lengths of flood records.

Effect on accuracy of flood estimation of errors in extracting catchment characteristics from maps - Task 3

IH has no plans formally to study these effects on a national scale but would be glad to cooperate in what could be an interesting student project.

Work has been done at Northumbrian Water Authority to compare flood estimates by the rainfall runoff method using climate parameters derived (a) by direct measurement from maps in FSR Vol 5 and (b) by interpolation in a 10 x 10 km grid digitised from the maps. As an example, the mean annual flood for 47 catchments in northeast England and the use of the grid introduced a standard error of 5.4%. (Further details from Dave Archer, Newcastle 843151.)

Extend range of catchments and events for detailed study - Tasks 4, 5 and 6

These tasks comprise a primary objective of a continuing MAFF commission at IH. Extra catchments have been identified for study and many more events analysed. Progress is, however, likely to be slow given the current difficulty in persuading authorities to extend - or sometimes even to maintain - their networks of flow gauging stations or raingauges. Much hope is pinned on widespread acceptance of the Representative Basin Network - a set of about 200 catchments designed to provide good quality data from a wide range of catchment types.

The network will, wherever possible, include experimental catchments set up by universities and others. Several such experiments have been designed to study the effects of change in land use such as those listed under Task 5. IH are actively investigating:

> *urbanisation* (see Supplementary Report No 5) - continuing to monitor catchments undergoing progressive urbanisation; using more sophisticated models designed to reflect the effect of the location (as well as the extent) of any urban development inside a catchment area; continuing to monitor the statistics of flood flows from catchments in various stages of urbanisation.
>
> *drainage* - conducting small-scale experiments (*Ref:* M Robinson & K Beven, *J. Hydrol. 1983 - in press*) and collating data from the experiments of others. Such studies have shown that the effects of drainage can be significant at the local scale and may be broadly related to soil type. Further work is needed to establish the importance of the effects on flood hydrographs for catchments of the size considered in the FSR. As a first step, MAFF records of the location of field drainage are being analysed with the aim of formulating a new catchment characteristic.

Afforestation - studying catchments undergoing afforestation (*Ref:* IH Report No. 73, 1980) and by comparing flows from forest and unforested areas (*Ref:* 'The Plynlimon Report', edited by M D Newson, *in press*). Traditionally, forests were assumed to reduce flood risk but recent work has demonstrated that forest ditches - dug before planting - can have the opposite effect in a young plantation. Work is progressing to establish the relative importance of ditches and trees on storm runoff from mature forests.

The variation of percentage runoff (or runoff coefficient) between catchments is clearly related to the permeability of the soil and the slopes of the drainage paths on and just below the surface. Quantificiation of these effects on a catchment basis is attempted, in the FSR, by means of the catchment characteristic, SOIL. SOIL indices are derived from a map of five soil classes produced by the Soil Surveys (see Supplementary Report No. 7). Subsequently the Institute is using the further data mentioned above with the aim of improving the map by feedback of observed percentage runoff data. This approach is being extended to include the base flow index (BFI) (available at a larger number of sites than percentage runoff - see Supplementary Report No. 13) for classifying soil associations. Progress in this area is most advanced in Scotland where a map of BFI is in preparation.

Regarding the variation of percentage runoff between events, the general effects of CWI (dominated by SMD) are clear; when it is wet beforehand there is more runoff. Experience has shown, however, that the relationship between percentage runoff and CWI is poorly defined on individual catchments and quite variable between catchments. It is thought that these uncertainties may be affected by poor estimates of average SMD in the catchment area. As a first step, therefore, IH intends to rework the FSR (and subsequent) events using an improved procedure for computing areal SMD.

Although these tasks have been interpreted with particular reference to the rainfall/runoff approach to flood estimation, it is advantageous also to extend the range of catchment types used to calibrate the $\bar{Q}$ approach. In this context, the European Flood Study should provide useful data from many extra catchments some of which show a more extreme range of catchment characteristics.

Flood estimation methods for small catchments - Task 8

IH has reviewed FSR procedures for all suitable gauged catchments with areas less than 20 km^2 but could not improve upon the standard recommendations (Supplementary Report No. 6). Poots & Cochrane* produced an alternative equation for $\bar{Q}$ prediction by reanalysing the small catchment subset from the FSR. Being a little simpler than the FSR equation, it was claimed to be appropriate for quick or less crucial estimates. However, users should refer to the discussion of the paper for elucidation of an error that appears in the design factor.

There has been some work at IH to show that standardisation on a 1 hour unit hydrograph characterised by its time to peak can lead to anomalies in applying the rainfall/runoff method to synthesize floods on very rapidly responding small catchments. This is not a problem to undermine the FSR procedure but some users might like to pursue the topic in which case please contact Duncan Reed and refer to Informal Note 63.

Extending time series by correlation with non-hydroloical data - Task 9

Although the UK has a richly documented history enabling some headway to be made with the construction of flood event records for individual sites (see Supplementary Report No. 4), the extension of data by means of tree rings or mud varve analysis appears to be an unlikely prospect given our generally rather small and relatively responsive river basins. Certainly, past efforts using tree ring width and density proved fruitless; typical correlations were around 0.6 which is too low for practical data extension. There is no current research programme in this area at IH although research is being conducted at the Climatic Research Unit, University of East Anglia.

**Proc. Instn. Civ. Engnrs. Pt 1,* 1976, 66 (Nov), 663-666 and Discussion, 1980, 68 (May), 315-322.

Changing flood risk within the year - Task 10

Supplementary Report No. 2 looked at flood estimation for return periods less than a year but that study did not consider specific periods within the year. Current work at IH is studying the date of occurrence of the annual maximum flood and how this varies regionally and with catchment characteristics. Results to date show that there is a national trend with catchments in the north and west tending to flood earlier in the season than catchments in the south and east. The primary control is thought to be soil moisture deficit with the return to zero SMD occurring about 60 days earlier in the north and west than in the south-east. A second order effect may be the influence of catchment response time with steep or urbanised catchments responding to high short duration rainfalls which tend to occur in the summer.

Frequency of joint tidal and fluvial events - Task 11

A paper describing work done at IH is to be given at the IUGG conference in Hamburg (1983) and will be followed by a supplementary report. The results point to the need for a correct interpretation of probabilities - certainly one cannot combine, for example, the 10 year return period flood and the 5 year return period tide and expect a 50 year return period level. The method, while perfectly general, cannot be reduced to simple 'regression equation' format and does require a model of estuary behaviour as well as the statistical descriptions of river flows and tide levels.

Rainfall statistics - Tasks 12 to 17 inclusive

The rainfall statistics presented in the FSR were developed by the Meteorological Office. They are not pursuing any research directly in the areas covered by the five tasks but are devoting considerable resources to the checking, possible further calibrating, archiving, and methods of analysis of radar derived rainfall estimates. A new radar-based methodology, involving statistics of peak intensity, areal extent, and velocity, could side-step several of the issues raised in the listed tasks such as that of raingauge independence and areal reduction factors.

There are, however, significant problems remaining. For example, the radar echo v. rainfall intensity relationship has to be calibrated in real time using telemetering raingauges and/or retrospectively using whatever gauge data are available: the extent to which this relationship can be safely extrapolated to gaugeless areas of high intensity (Task 14) is still uncertain.

Pending the development of the radar-based methodology, IH is proceeding with an improved method for combining daily raingauge totals and hourly rainfall data to provide a consistent estimate of an areal average rainfall profile (S. B. Jones, Institute of Hydrology Report - *in press*).

Although the radar-based work may eventually help with some of the topics listed under Task 16, it is thought that the early introduction of more realistic profiles and areal varia (topic b) into maximum storm construction would be widely welcomed; resources permitting, IH would like to do some work in this area.

Of the many other rainfall tasks identified in the Table, No. 17 would make a suitable project for a final year undergraduate and No. 12 offers scope for post-graduate research.

Frozen ground and snowmelt - Tasks 7 and 18

Neither the Meteorological Office nor the Institute have research in progress relating to frozen ground *per se*. Despite the belief in some quarters that, in our climate, the ground is rarely frozen beneath a large snowpack, the effects of frozen ground are usually thought to be associated with floods arising mainly from snowmelt rather than exclusively from rainfall.

This illustrates a considerable gap in our knowledge of physical processes but one which is very difficult to fill without an extensive programme of instrumentation and monitoring.

Recent work by Archer* (Northumbrian Water Authority) has helped to emphasise deficiencies in the FSR approach to snowmelt computation but there is no current Meteorological Office involvement in any of the topics of Task 18. At IH, both modelling and field work (Scotland and Norway) are being undertaken to improve understanding of the processes affecting snowmelt, the routing of melt through the snowpack and through or over the soil. This work is associated with the development of physically-based distributed models that permit the effect of the spatial pattern of snowpack depths and melt rates to be taken into account. These models remain research tools at present but should have some spinoff for engineering practice in the future.

Information exchange in the future

That completes the account of FSR-related research currently in progress and known to IH. As mentioned in the introduction, it could be useful to extend the scope of any follow-up articles and the Institute's probable involvement with a newsletter for the British Hydrological Society (see later) could provide the means.

WALLINGFORD TAKES OVER NATIONAL WATER DATA ARCHIVE

With the transfer, to the Institutes of Hydrology and Geological Sciences, of national surface water and groundwater archiving responsibilities, the opportunity presents itself for research staff to make more effective use of, and be better informed about the quality of, their data sources. The surface water archive will routinely collect mean daily flow data from about 650 catchments. However, the exercise will embrace the expansion of two other archives with more particular relevance to Flood Studies research:

'Peaks over threshold' The collection of all relevant flood peak data up to, say, 1980 will almost double the average record length (to about 20 years) and thereby increase the confidence in individual derived flow frequency curves. These data will be gathered either from a continued programme of chart microfilming or abstracted directly from the gauging authority's 15 minute archive.

'Flood events' The representative basin network is a subset of about 200 catchments for which more detailed data relating to flood events will be gathered. The network includes many of the 'unit hydrograph' catchments from the FSR but there has been a deliberate attempt to bring in extra catchments from previously unsampled regions. The main requirement, yet to be met in many cases, is for well sited and well run recording raingauges. Tasks 4, 5 and 6 in the previous section indicate the way in which the enlarged event archive will be exploited.

The system of regional representatives (most of whom will also be research hydrologists) to be used in running the surface water archive will ensure that problems with data are well appreciated and should help to provide a more regular two-way exchange of information.

COMPUTERS, ARCHIVES, AND MAPS

As more and more data arrive in one computer archive or another it becomes increasingly useful not simply to retrieve the information (that much is vital) but to do so as flexibly as possible and in conjunction with other data which may be archived elsewhere. Such retrieval can be used to build overlaid displays on a graphics terminal (which can in turn guide further requests for information) or plots for reports and displays.

At the Institute of Hydrology there is considerable interest in developing this type of facility as an aid to the analysis of flood events and rainfall-runoff modelling in general. In addition to the national surface water archive of daily flows, data libraries hold: flood event data (flow hydrograph, recording raingauge records, SMD); all daily rain data from 1961; catchment boundaries; all instantaneous peak flows over a given threshold; the coastline, hydrometric area boundaries and river network as digitised from the 1:250,000 map; the river network from the 1:50,000 series (parts only); the soils map; details of gauging stations; a catalogue of recording raingauges in the UK.

**Proc. Instn. Civ. Engnrs. Pt. 2*, 1981, 71 (Dec), 1047-1060.

THE EUROPEAN FLOOD STUDY

Digital conversion of soils and land use maps has also been adopted as a key feature in the application of FSR methodology to much of Northern Europe. 1200 catchments from France, Denmark, Germany and the Benelux countries will provide an expansion in the range of catchment types and, it is confidently expected, improved insight into the structure of derived regression equations. The enlarged data set should, for example, help understanding of the controls on exponent values, regional groupings, and the optimum choice of catchment characteristics in different circumstances. This will in turn guide development towards any revised methodology for use in the UK (see also the previous discussion of Task 1 from Table 1).

The project, which is confined to mean annual flood and flow frequency topics, is currently at the data archiving and map digitising stage with analyses to start during late 1983.

FLOOD FREQUENCY CURVES - WORLDWIDE

Not content with plundering the European mainland for valuable data, Institute hydrologists are combing the world for long records of annual maximum floods. The objective is to relate the shape (i.e. slope and curvature) of regional flood frequency curves to climatic and physiographic characteristics of the region. Preliminary results have been presented by Sutcliffe* but a more detailed report is in preparation. It should prove useful to FSR users with possible overseas application for the methodology. To add to the request for casebook material below, it would be most helpful if overseas travellers could bring back more duty-free data to extend the scope of the study still further. All offers to Frank Farquharson.

CASE STUDIES OF FSR USAGE

The Institute of Hydrology often receives telephone or written enquiries about one or other of the several applied hydrological techniques which have emerged in recent years. Some enquiries develop into a 'consultancy job' and examples of these are appended (Table 2). Others are dealt with by a single exchange of correspondence. Attempts are made - by internal discussion and by formalising advice on some topics (e.g. in Supplementary Reports) - to ensure a consistent approach but it might be useful to consolidate these by producing a casebook. It could include examples of problems tackled not only at the Institute but by all users. If anybody would be willing to contribute an interesting design problem connected with flood hydrology but outside the straightforward application of the published guidance, please send brief details to Mike Lowing. To ensure confidentiality, there is no need to give precise locations or actual figures; it is the tailoring of techniques to problems that is of interest.

Table 2 lists some of the case studies involving flood design carried out by the Institute of Hydrology in conjunction with a number of consulting engineering firms, with regional water authorities, and on behalf of UK or overseas government organisations. Although the UK studies have been based on FSR recommended procedures those overseas have often involved adaptation of the general FSR approach to the local situation. Local data (Supplementary Report No. 13) will have been used wherever possible but, due to cost restraints, not always to the fullest extent.

A REVIEW OF RECENT DESIGN GUIDES

The last decade has seen the appearance in the UK of a number of design guides in engineering hydrology. Table 3 is a list, in data order of publication, which attempts to show how they relate to each other.

*Sutcliffe, J. V. Use of the Flood Studies Report overseas. "Flood Studies Report - Five Years On", ICE Conference, Manchester, 1980, 7-10.

TABLE 2 Case studies, by IH in period 1974-82, using *FSR* recommendations or principles

Location	*Client*	*Brief description*
In the UK		
River Lagan, N. Ireland	Hydraulics Research Station	Provision of design hydrographs for input to flood routing model. Advice regarding the effects of future land drainage operations on design inputs.
Cwmystradllyn, North Wales	Howard Humphries & Partners	Derivation of PMF for reservoir site and comparison with 'catastrophic' flood from 1933 ICE Guide.
Upper Derwent Valley	Derwent Valley Water Board	Synthesis of flood hydrographs (through cascade of reservoirs) which might have followed July 1973 event if catchment had been wet (rather than dry) and reservoirs full (rather than depleted). Estimation of PMF.
Ifield Mill Pond, Crawley	Rofe, Kennard & Lapworth	Derivation of flood frequency relationship and PMF. Consideration of effects of further urban development.
Ardingly, Sussex	Rofe, Kennard & Lapworth	Flood frequency curve and design hydrographs at proposed dam site.
Lower River Avon, Bristol	Sir Alexander Gibb & Partners	Application of unit hydrograph technique to investigate influence of proposed barrage on flood regime.
Lincoln	Anglian Water Authority	Hydrological aspects of flood storage pond. Investigation of probability of volume floods for design of off-stream flood storage ponds.
River Nene	Anglian Water Authority	Application of statistical and unit hydrograph approach for estimating hydrological inputs from gauged and ungauged catchments to hydraulic model of River Nene. Full use made of Base Flow Index relations.
Hadlow, Kent	Southern Water Authority	Flood estimation on very small (approx. 1 km^2) catchment based on an analysis of local data from a much larger catchment.
Warminster, Dorset	Lemon & Blizard	Use of unit hydrograph/losses model to synthesise the dominant slow response of a small permeable chalk catchment.
Overseas		
Botswana	Sir Alexander Gibb & Partners (Africa)	Assessment of the yield provided by increasing the capacity of Gaborone reservoir and the estimation of flood magnitude. Study of the implications of prolonged droughts.
Indonesia	Sir Alexander Gibb & Partners	Water resource and flood studies for possible hydropower stations on the Ayung and Balian catchments on Bali.
Iran	Sir Alexander Gibb & Partners	Analysis of the frequency of flooding in the Khuzestan plain as part of a project to define flood protection measures around oil installations.
		Rainfall duration-frequency analysis for the Tehran sewerage project.
	Trevor Crocker & Partners	Flood estimates for bridge design on the Qom to Isfahan route.
Morocco	Sir Alexander Gibb & Partners	Water resource and flood studies for a dam on the Oued Ksob. Comparisons with 'GRADEX' approach.
Mozambique	Rendel Palmer & Tritton	Studies of the potential for flood alleviation and flood warning on the Lower Zambesi including a review of the operation of Cabora Bassa.
Nigeria	Milton Keynes Dev. Corp.	Design flood estimates for civil works in the new federal capital area.
	Scott Wilson Kirkpatrick & Partners	Water resources and flood studies for various water supply schemes.
		Flood estimates for a road bridge on the river Rima at Sabon Birni.
	Ward Ashcroft & Parkman	Flood estimates for a dam on the river Aiye at Yekemi.
		Review of the hydrology of the Kaduna basin leading to generalised estimates of flows at ungauged sites from climate, topography and soils information.
		Flood hydrology of the Gongola catchment including an unusual application of a unit hydrograph to a 25-day 'design storm'.
South Korea	Binnie & Partners	A countrywide study of all hydrological data leading to the development of a prediction equation for $\bar{Q}$ and regional flood frequency curves.
Sri Lanka	Sir Alexander Gibb & Partners	Water resource and flood studies for the Victoria project; a major hydropower and irrigation scheme within the Mahaweli development programme.
	Sir William Halcrow & Partners	Review of spillway design flood for the Kotmale dam.
Indonesia	Overseas Dev. Administration	For Java and Sumatra, the development of flood frequency curves.
Brazil	UNDP	For Rio Grande do Sul State, the development of a flood frequency curve.

TABLE 3 Design guides in flood hydrology: 1973-83

	Date	*Title*	*Published by*	*Comment*
1	1973	The estimation of flood flows from natural catchments	TRRL, Rep. LR 565	Produced a formula, applicable to certain small catchments, for Q_T in terms of catchment size and slope, annual average rainfall and 'Bilham' rainfall. Formula calibrated with data from 5 catchments.
2	1975	Flood Studies Report (5 volumes)	NERC (obtained from IH)	A compendium of methods for Q_T estimation and full hydrograph determination on any catchment in UK. Includes major advance (by Met. Office) in computation and presentation of rainfall statistics to supersede the Bilham formula. Rainfall estimates use several maps of key variables. There is also a national 'soils' map for use in estimation procedure. Main map series at 1:625,000 scale. Methods based on data from about 700 catchments.
3	1975	Reservoir Flood Standards	ICE	A discussion paper only - issued for consideration alongside the *FSR* at *FSR* Conference.
4	1975	Flood Studies Conference - Proceedings	ICE	Summary papers by *FSR* authors. Includes first appearance of a map of the RSMD variable but this map is more conveniently available in Nos. 8 and 9 below.
5	1975	Inspection, Operation and Improvement of Large Dams. Conference Proceedings	BNCOLD, and Newcastle University	Included a number of papers on flood analysis. First appearance of a formula for rapid calculation of PMF peak flow (Paper 4.7). Further discussion of No. 3 above.
6	1976	A guide for engineers to the design of storm sewer systems	TRRL Road Note 35	Replaced earlier (1963) version of note. Used some aspects of *FSR* rainfall statistics.
7	1977	Flood Studies Supplementary Reports begin	IH	First batch included (No. 3) a report of the Birmingham seminar - the first real opportunity for *FSR* users to discuss problems.
8	1978	Methods of flood estimation - a guide to the *FSR*	IH	Colloquially referred to as the 'slim guide' (50pp). Concentrates on the two main procedures of *FSR* Vol I. Adds the RSMD map from No. 4 and the PMF formula from No. 5.
9	1978	Floods and reservoir safety: an engineering guide	ICE	Finalised following discussion of No. 3. Intended as formal replacement of ICE report on Floods in Relation to Reservoir Practice (1933 & 1960). Includes a graph based on a simplification of the PMF quick method and also shows the RSMD map.
10	1978	Estimation of run-off potential of river catchments from soil surveys	Soil Survey (Rothamsted) Special Survey No. 11	Background to production of 'soils' map in No. 2. Included a revised (and coloured) version of the map (for England and Wales only) at a scale of $1:10^6$.
11	1978	Flood prediction for small catchments	IH Flood Studies Supp. Rep. 6	Presented simplified formula for rapid estimation of $\bar{Q}$ on small catchments but recommended continued use of existing *FSR* method.
12	1978	Revised WRAP (Soils) map	IH Flood Studies Supp. Rep. 7	A revised version of the *FSR* soil map (at 1:625,000) based on No. 10 but covering all UK and extending to include urban areas.
13	1979	Design flood estimation in catchments subject to urbanisation	IH Flood Studies Supp. Rep. 5	A major extension of *FSR* methodology concerning catchments with a significant existing or planned urban fraction.
14	1979	Design flood estimation for bridges, culverts and channel improvement on small rural catchments	ICE Proc. Pt 1 Nov. - see also discussion in May 1980	Similar equation to that produced in No. 11.
15	1980	Guide to the design of storage ponds for flood control in partly urbanised catchment areas	CIRIA Tech. Note 100 (draft for discussion)	Hydrological content very similar to No. 13 except for different treatment of the unit hydrograph shape. Optional set of maps (including soils) is a subset of those in No. 17.
16	1981	Manchester Conference (1980) on Flood Studies Report - Five Years On	ICE	Review papers by *FSR* authors and others. The main forum for serious discussion of *FSR* limitations. Conclusions and recommendations for further research published in ICE Proc. Pt. 1 (Nov) 1981, 833.
17	1981	*FSR* rebound with minor corrections	(NERC) IH	The revised soil map (No. 12) now included (with further minor amendments) in the maps volume of the Report itself.
18	1981	Design and analysis of urban storm drainage (4 vols)	National Water Council	Intended as modern replacement and considerable enhancement of No. 6. Hydrological input from same 'stable' as *FSR*. Includes *FSR* rainfall statistics procedures in full. Maps (at $1:10^6$) are easier to use than *FSR* equivalents. Soils map has the same content as the version now in the *FSR* (No. 17).
19	1983	Various titles	Further batch Supp. Reps from IH	More design guidance covering the use of local data, the combining of flood frequency curves derived in different ways, spillway design when a number of reservoirs are in series, etc.

BRITISH HYDROLOGICAL SOCIETY

At the time of writing (June 1983), discussions on forming a British Hydrological Society are virtually complete. All those who returned the questionnaire sent out at the end of last year will have received a report from the Initial Planning Group which was formed in Exeter in July 1982.

The proposal is for an independently controlled society with administration supported by the Institution of Civil Engineers but with 'scientific' links to the Royal Society and with the Institute of Hydrology. IH input will be mainly the editing of a regular newsletter. It is hoped that the new Society will have a strong regional structure and will hold joint meetings with other groups and societies active in related subjects. An inaugural meeting is planned for early November at the Royal Society.

Enquiries to the following members of the Initial Planning Group:

Mike Mansell Moullin; Janet Bonthron - Binnie and Partners (01 222 7755)

Mike Lowing; Elizabeth Morris - Institute of Hydrology (0491 38800)

Des Walling - University of Exeter (0392 77911)

CORRIGENDUM TO FSR VOLUME I

Interpolation for estimated maximum rainfalls

One error in the Flood Studies Report has significance to users concerned with maximum flood calculations. This is referred to in the preface to the second binding of the FSR (1981). The item to be corrected is the method of interpolation for estimated maximum rainfalls between 2 and 24 hours. II.4 recommends interpolation on linear-log paper whereas Step 11M of the design procedure given in I.6.8.3 mistakenly prescribes the use of log-log paper. The following corrections should be made in manuscript:

p.473/p.33* , Sentence below Table 6.25.
Delete "double log paper", insert "linear-log paper".

p.474/p.34* Figure 6.68
Cross out numbering on y-axis. Insert: "Linear scale recommended on y-axis. See II.4.3.4 and II.8.3.4".

The effect of the correction is to raise interpolated values; the departure is greatest for durations of about 6 hours. For a typical Pennine catchment, the 6 hour rainfall interpolated using linear-log paper is about 5% higher than using log-log paper. In low rainfall areas the discrepancy is less. However, in very high rainfall areas (ie. mountainous regions in Western Britain and Western Ireland) differences of 10 to 15% will be found.

It should be noted that the correction makes little or no difference for durations close to 2 or 24 hours.

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* The first reference is to FSR Volume I, the second is to the report: "Methods of flood estimation: a guide to the Flood Studies Report" which reproduces the design procedure for maximum flood estimation.

Flood Studies Supplementary Report No 16 Dec 1985

The FSR rainfall-runoff model parameter estimation equations updated

1. Introduction

Since publication of the Flood Studies Report (FSR) and partly as a response to comments made at the 1975 Flood Studies Conference the collection and analysis of event data have continued. It is now possible to review the rainfall-runoff method of flood estimation and in particular the parameter estimation equations. At this stage in the life of the FSR it was thought best to introduce improvements whilst maintaining as much as possible of the existing methodology; if changes were to be made they should be easily accommodated within the present framework. While this restriction implies a review of the parameter estimation equations only, the opportunity has been taken to consolidate many of the recommendations concerned with the rainfall-runoff method previously published in the FSR and Flood Studies Supplementary Reports (FSSRs). In addition, the choice of dependent and independent variables used in the regression analysis has been reviewed in an attempt to ease application of the method and solve problems encountered in application under extreme conditions. This report summarises the results of the review and the new recommendations but does not give a full account of the analyses; such an account may be found in Institute of Hydrology Report No. 94 (Boorman, 1985).

2. Data

The collection of new data was aimed at increasing both the number of events from the original set of catchments and also from extra catchments of types not represented in the FSR data set. Approximately 1000 new events were selected thereby increasing the total number of events available for analysis to over 2500. All of the event data were checked and as a result many events had to be rejected (including some of the original ones). In addition, the values of catchment descriptors (such as AREA and SAAR) were checked. One effect of this checking was to change the derived parameter values and the previously published catchment characteristics for some basins. Tables containing the validated data are to be found in IH Report 94.

3. Parameter estimation equations

3.1 Unit Hydrograph

The internal unit hydrograph relationships were supported by the new analyses and the recommendation to use a simple triangular unit hydrograph is maintained. However, to estimate the key unit hydrograph parameter, its time to peak, a variation of the existing procedure is recommended in which the time to peak of the T hour unit hydrograph, Tp(T) (T being the data interval) is estimated via the time to peak of an equivalent instantaneous unit hydrograph, Tp(0). For those unfamiliar with this concept it should be noted that whereas the T-hour unit hydrograph represents the response to a uniform input (rainfall) occurring over a T-hour period, the instantaneous unit hydrograph is the response of an equal but instantaneous rainfall occurring at the start of the period. Tp(0) is to be estimated from

$$\mathrm{Tp}(0) = 283.0\ \mathrm{S1085}^{-0.33}(1+\mathrm{URBAN})^{-2.2}\mathrm{SAAR}^{-0.54}\mathrm{MSL}^{0.23}$$

Users familiar with the FSR equation for Tp will note that SAAR replaces RSMD and thus saves some effort in calculating this variable. The data interval is then taken as a convenient value (eg 0.5, 1 or 2 hours) such that

$$T \cong \mathrm{Tp}(0)/5$$

Tp(T) is then calculated from

$$Tp(T) = Tp(0) + T/2$$

Obtaining the estimate of Tp(T) in this way is preferred as it avoids problems encountered on fast responding catchments (see Reed, 1985, for discussion).

Tp(T) is then used exactly as in the FSR procedure to calculate Qp (the unit hydrograph peak ordinate), Tb (the unit hydrograph time base) and D (the duration of the design storm).

3.2 Percentage Runoff

In the FSR, percentage runoff is estimated as the sum of a standard term and a dynamic term. The new recommendation is to remove the urban component from the standard term and to make a separate, and more realistic, allowance for the increased response from urbanised areas.

The percentage runoff from rural areas, PR_{RURAL}, is estimated from

$$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$$

where SPR is a standard percentage runoff dependent only on the five soil fractions S1 S2... S5

$$SPR = 10\ S1 + 30\ S2 \quad 37\ S3 + 47\ S4 + 53\ S5$$

DPR_{CWI} is a dynamic component of percentage runoff reflecting the increase in percentage run-off with catchment wetness

$$DPR_{CWI} = 0.25\ (CWI - 125)$$

and DPR_{RAIN} is a second dynamic component that increases percentage runoff from large rainfall events

$$DPR_{RAIN} = 0.45\ (P - 40)^{0.7} \quad \text{for } P > 40 \text{ mm}$$

$$= 0 \quad \text{for } P \leq 40 \text{ mm}$$

Once PR_{RURAL} has been found it is adjusted for urbanisation using

$$PR_{TOTAL} = PR_{RURAL}\ (1.0 - 0.3\ URBAN) + 70\ (0.3\ URBAN)$$

This equation represents those areas mapped as urban development on small scale (eg 1:50,000) maps as being only 30% impervious; this impervious area generates 70% runoff while the remaining 70% of the area responds as the natural part of the catchment.

By comparison with the SPR and PR equations given in the FSR it is seen that SPR values are slightly decreased for soil type 1 catchments and increased for soil type 5 catchments. The dynamic terms are larger, but the rainfall term is only applicable to substantial rainfall events. The equations modelling generation of runoff from urban areas now reflect the mixed natural and impervious areas that occur within urban areas and therefore the urban effect is dependent on the soil type on which the development is sited.

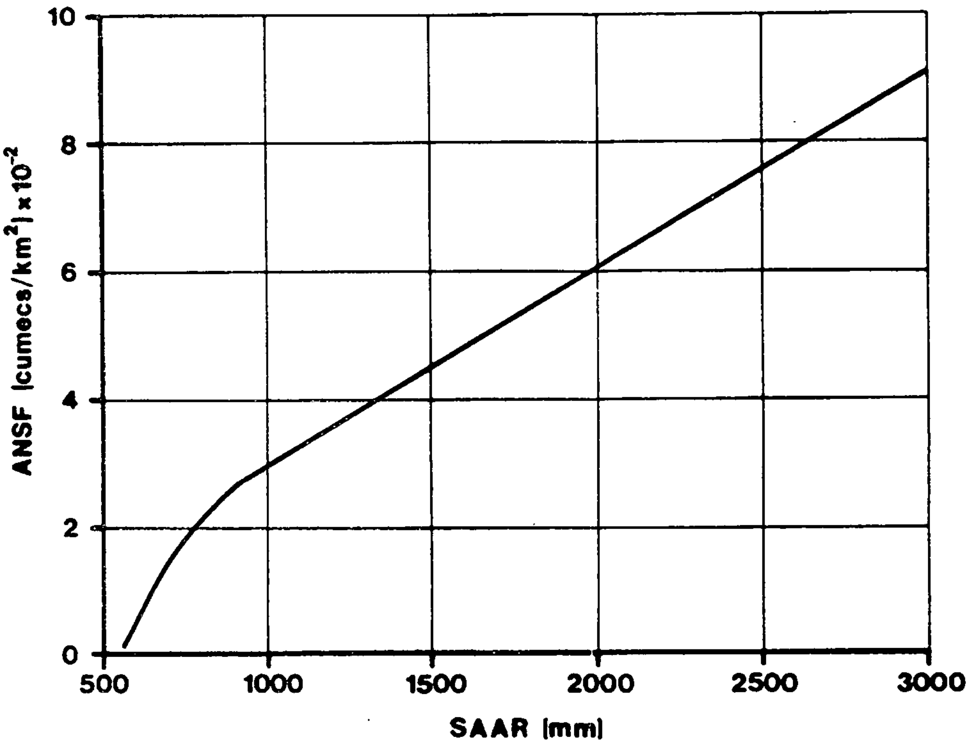
3.3 Baseflow

Of relatively minor importance in flood estimation is the addition of baseflow to the response hydrographs. ANSF, the baseflow in cumecs per square kilometre that is added to each ordinate of the response hydrograph, is given by

$$ANSF = [33\ (CWI - 125) + 3.0\ SAAR + 5.5] \times 10^{-5}$$

By comparison with the ANSF equation in the FSR it is seen that SAAR again replaces RSMD. Since in design use CWI is estimated from SAAR, ANSF is solely dependent on SAAR and the value obtained from the equation can be checked against the graphed relationship in Figure 1.

Figure 1. **Graphical representation of ANSF-SAAR relationship for design use. (Applicable to T-year case but not PMF)**



4. <u>Accuracy of Estimates</u>

Estimates of Tp(T), PR and ANSF obtained using the above equations are, on average, only very slightly more accurate than those obtained using the FSR procedures. Users are reminded of the usefulness of local data in refining flood estimates. A discussion on the incorporation of such data is to be found in FSSR 13 together with more formalised advice that remains apposite. In transferring values of Tp, however, it should be noted that Tp(0) is the value to be adjusted (not Tp(T)). Calculating SPR from event data will require inversion of the equations presented in this report but otherwise the principle of the method is as described in section 3 of FSSR 13.

Two specific types of data that may be available are to be used with revised equations. If rainfall and stage data are available they can be used to find LAG from which Tp(0) may be estimated from

$$Tp(0) = 0.604\ LAG^{1.144}$$

Daily mean flow data can be used via the Low Flow Studies (Institute of Hydrology, 1980) index BFI using

$$SPR = 72.0 - 66.5\ BFI$$

which is a revised version of the equation that appears in FSSR 13.

5. <u>Application to urban catchments</u>

The revised equations presented above are recommended in place of the original FSR equations and those of FSSR 5 for application on urbanised catchments (ie catchments with URBAN >0.25). The revised equations do not alter the FSSR 5 recommendations that on urban catchments the design storm depth has the same return period as the design flood, and that this is distributed in time using the 50% summer profile.

In addition the new percentage runoff equation also affects the statistical approach of flood estimation for urban catchments as presented in FSSR 5. On catchments outside the Thames, Lea and Essex area ([1]), the mean annual flood on an urban catchment is estimated by a 6-variabl equation (which contains no URBAN term), to which a correcting factor is applied for urbanisation. This recommendation (FSSR 5, section 5.1) is maintained although the percentage runoff term used in obtaining the factor should come from the revised equations given in this report. Note that the rainfall term in the percentage runoff equation does not contribute in the mean annual flood event since the storm depth will generally be less than 40 mm.

6. Estimation of Probable Maximum Flood (PMF)

Two points are worth noting when the new equations are used to estimate PMFs. In PMF estimation some aspects of the design procedure are modified to allow for the unusual conditions that may exist under such circumstances. One of these modifications is to reduce Tp by one third; in the revised procedure this adjustment should be made to Tp(0). Secondly, design CWI the PMF case is not solely a function of SAAR so that Figure 1 is inapplicable.

7. Conclusion

It is pleasing to report that the review of the FSR equations contained in IH Report 94 concludes that FSR parameter estimation equations are not seriously deficient. However, revised equations are presented, and are now recommended for design use. In their derivation consideration has been given to easing problems encountered in deriving catchment characteristics and in applications under extreme conditions; The new equations, which are summarized on the following page, offer a consolidated set of guidelines updating recommendations previously published in the FSR and FSSRs.

References

Boorman D B (1985) 'A Review of the Flood Studies Report Rainfall-Runoff Model Parameter Estimation Equations' Institute of Hydrology Report No.94.

Institute of Hydrology (1980) Low Flow Studies, Institute of Hydrology, Wallingford.

Reed D W (1985) 'Extension of the S-curve method of unit hydrograph derivation' Proce. ICE, Part 2,79:193-201.

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[1] For catchments in the Thames, Lea and Essex area (region 6) the use of an adjustment factor applies only to catchments with less than 25% urbanisation, and to catchments which are at present substantially rural but where urban development is planned and the post-urbanisation flood magnitudes are to be estimated. In such cases the mean annual flood is estimated using the national equation with the same multiplier as for region 5 (ie 0.0153). For substantially urbanised catchments the Thames, Lea and Essex equation for mean annual flood (which does contain an URBAN term) should be used without adjustment. See FSSR 5 for a full discussion of the applicability of the mean annual flood estimation equations in this area.

SUMMARY

Equations

Unit hydrograph parameters

$$Tp(0) = 283.0\ S1085^{-0.33}\ (1+URBAN)^{-2.2} SAAR^{-0.54} MSL^{0.23}$$

$$Tp(T) = Tp(0)+T/2$$

$$Qp = 220/Tp(T)$$

Percentage runoff

$$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$$

where

$$SPR = 10S1 + 30S2 + 37S2 + 47S4 + 53S5$$

$$DPR_{CWI} = 0.25\ (CWI - 125)$$

$$DPR_{RAIN} = 0.45\ (P - 40)^{0.7} \quad \text{for } P > 40 \text{ mm}$$

$$= 0 \quad \text{for } P \leq 40 \text{ mm}$$

$$PR_{TOTAL} = PR_{RURAL}\ (1.0 - 0.3\ URBAN) + 70\ (0.3\ URBAN)$$

Baseflow

$$ANSF = [33(CWI - 125) + 3.0\ SAAR + 5.5] \times 10^{-5}$$

Local data equations

$$Tp(0)=0.604\ LAG^{1.144}$$

$$SPR = 72.0 - 66.5\ BFI$$

Notation

ANSF	baseflow (m^3s^{-1}/km^2)
BFI	baseflow index
CWI	catchment wetness index (mm)
DPR_{CWI}	dynamic contribution to percentage runoff from CWI
DPR_{RAIN}	dynamic contribution to percentage runoff from rainfall
LAG	lag between centroid of rainfall and centroid of hydrograph peak (hours)
MSL	main stream length (km)
P	storm rainfall depth (mm)
PR	percentage runoff
PR_{TOTAL}	percentage runoff from both rural and urban areas
PR_{RURAL}	percentage runoff from rural areas
Qp	unit hydrograph peak ($m^3s^{-1}/100\ km^2$)
RSMD	effective 1 day rainfall of 5 year return period (mm)
SAAR	standard period average annual rainfall (mm)
SPR	standard percentage runoff
S1...S5	fraction of soil in WRAP classes 1 to 5
S1085	10-85% stream slope (m/km)
T	data interval (hours)
Tp	unit hydrograph time to peak (hours)
Tp(0)	time to peak of the instantaneous unit hydrograph (hours)
Tp(T)	time to peak of the T-hour unit hydrograph (hours)
URBAN	urban fraction

Flood Studies Supplementary Report No 17 Dec 1985

A localised re-interpretation of the WRAP (Soil) map

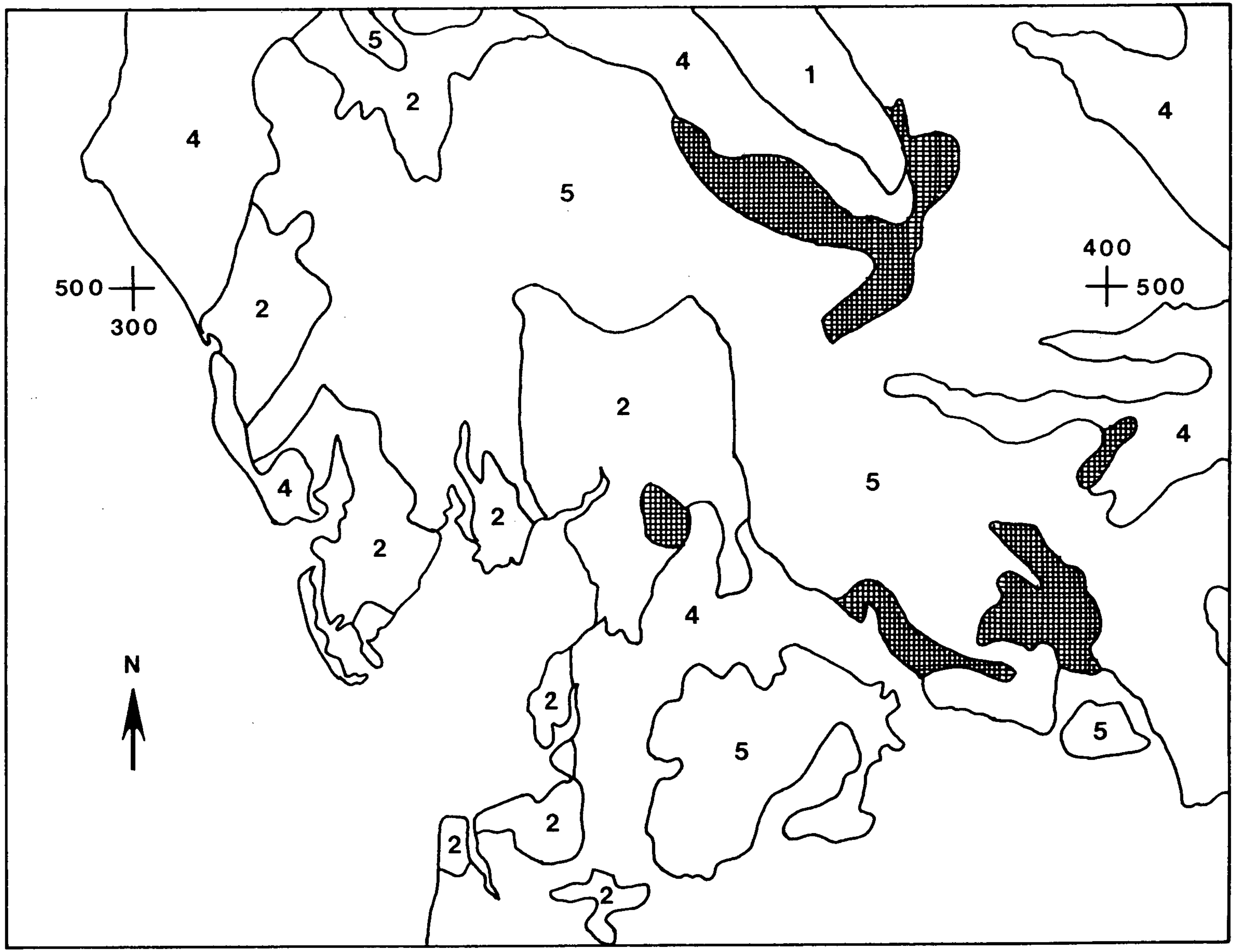
During analysis of the percentage runoff data collated for the review of the Flood Studies Report rainfall-runoff method parameter estimation equations (FSSR 16), a group of catchments on the Carboniferous Limestone of north-west England stood out as having very much larger percentage runoffs than would be expected from their soil classificiation. One catchment located on soil type 1 yielded percentage runoffs well over 50% (the highest being 75%). The catchments fell wholly or partly within a single soil association. The classification of this soil as type 1 with its rapid permeability rates and absence of an impermeable layer is in accordance with the WRAP (winter rainfall acceptance potential) scheme given in the FSR. However, catchment response is very rapid as a result of both the low soil moisture storage in these shallow calcareous soils and the quick response via the fissure permeable limestone. This has been substantiated by percentage runoff data and other evidence (Gustard, 1981) which suggest that this soil association's hydrological response is as from a type 5 soil. It is therefore recommended that this soil association be re-interpreted as type 5. The WRAP map has been redrawn for the area concerned and is shown in Figure 1 overleaf. Further details of this re-interpretation are given in Boorman (1985).

References

Boorman D B (1985) A review of the Flood Studies Report rainfall-runoff model parameter estimation equations. Institute of Hydrology Report No.94.

Gustard A (1981) The hydrological response of two upland catchments: implications for flood estimation. PhD Thesis, Lancaster University.

Figure 1. Areas of Soil 1 reinterpreted as Soil 5. Scale 1:625,000



Flood Studies Supplementary Report No. 18

October 1988

Collective risk assessment for sites sensitive to heavy rainfall

1. Introduction

Collective risk assessment for networks of impounding reservoirs in common ownership is one application of a procedure developed at the Institute of Hydrology. If the N sites under scrutiny are few and widely scattered, the effect of spatial dependence in heavy rainfall will be small and the collective risk of an exceedance of the T-year event can be calculated from:

$$r = 1 - (1 - 1/T)^N \qquad [1]$$

However, if the sites of interest are many and closely grouped it is essential to take spatial dependence into account when assessing the collective risk of a design exceedance. Such a method has been developed in research for the Department of the Environment's reservoir safety commission and is presented here for general use.

2. The risk assessment procedure

The collective risk of a T-year event occurring at one of a given network of N sites is estimated as follows.

STEP 1: Identify the N sites and their grid references (X_i,Y_i) in km units.

STEP 2: Calculate the mean intersite distance in km, $\bar{d}$, from:

$$\bar{d} = \frac{1}{N\ (N - 1)} \sum_i \sum_j \sqrt{(X_i - X_j)^2 + (Y_i - Y_j)^2} \qquad [2]$$

STEP 3: Estimate the area "spanned" by the sites, using the empirical formula:

$$\text{AREA} = 2.5\ \bar{d}^2\ . \qquad [3]$$

If the network is highly irregular, check that this provides a reasonable reference area by plotting a circle of radius $\sqrt{(\text{AREA}/\pi)}$ centred at the centroid of the N sites.

STEP 4: Estimate the duration, H hours, of heavy rainfall to which the individual sites are generally sensitive. Because of the nature of the daily rainfall data used to calibrate the spatial dependence model, it is necessary to convert this duration into units of rain-days. A precise value for this parameter is not crucial to the collective risk assessment and an appropriate value of D can be taken from Table 1.

Table 1

Storm duration, H, to which sites deemed sensitive (hours)	Duration, D, to be used in model (days)
H where H<15	H/18.0
15 - 22	1.0
22 - 33	1.5
33 - 53	2.0
H where H>53	H/24.0

STEP 5: Evaluate the equivalent number of independent sites, N_e, from the spatial dependence model:

$$\ln N_e = \ln N \,(a + b \ln AREA + c \ln N + d \ln D) \qquad [4]$$

where a, b, c and d are regional parameters defined by Table 2 and Fig. 1.

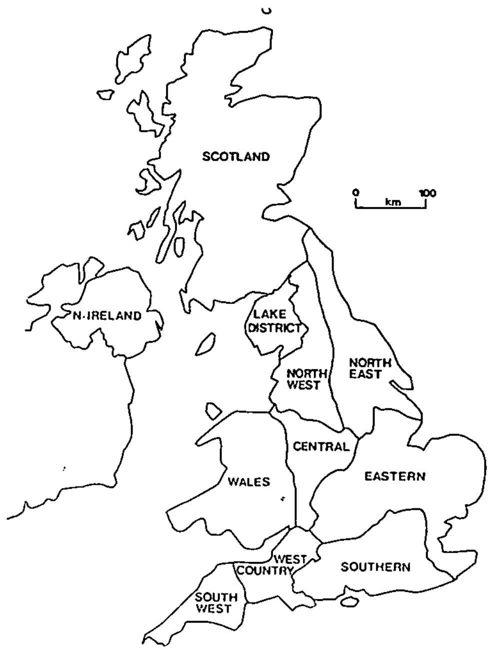
Table 2 Regional parameters for use in Equation 4

Region	a	b	c	d
North East	0.055	0.082	-0.058	-0.040
Eastern	0.0	0.091	-0.050	0.0
Southern	0.067	0.089	-0.032	-0.036
West Country	0.0	0.101	-0.085	0.0
South West	0.0	0.095	-0.058	0.0
Wales	0.097	0.085	-0.052	-0.035
Central	0.0	0.093	-0.048	-0.037
North West	0.069	0.091	-0.048	-0.055
Lake District	0.0	0.109	-0.076	-0.021
Scotland	0.188	0.073	-0.056	-0.029
N Ireland	0.0	0.086	-0.059	0.0
UK	0.081	0.085	-0.051	-0.027

STEP 6: The required collective risk of an exceedance of the T-year event at one of the sites is obtained from:

$$r = 1 - (1 - 1/T)^{N_e} \,. \qquad [5]$$

Figure 1
Rainfall regions



3. Examples

A: IMPOUNDING RESERVOIRS IN UPPER TAFF

Locations of 22 major impounding reservoirs in the headwaters of the Taff river basin are shown in Fig. 2. These are taken from the Register of Reservoirs compiled by the Welsh Office in 1984. An assessment is required of the likelihood of experiencing a 10,000-year flood at one or other of these reservoirs.

STEP 1: N = 22. The sensitive sites are defined by the grid references of the catchment centroids.*

STEP 2: The mean intersite distance, $\bar{d}$, from Equation 2 is 9.98 km.

STEP 3: Applying the empirical formula (Equation 3):

AREA = 2.5 $(9.98)^2$ = 249 km^2.

Evaluating the centroid of the 22 sites and constructing a circle of equal area, it is confirmed that the formula provides a reasonable estimate of the area spanned by the network.

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*When the network comprises many sites, the collective risk assessment is relatively insensitive to the detailed layout. In this example it would have sufficed to represent the location of each reservoired catchment by the grid reference of its dam rather than evaluating the catchment centroid. The resultant estimate of the area spanned by the 22 sites would have been reduced from 249 km^2 to 222 km^2, leading to only a small change in the collective risk assessment.

STEP 4: A typical design storm duration for these reservoirs is estimated to be 9 hours. From Table 1 this converts to a D value of 0.5 days.

STEP 5: Noting from Fig. 1 that the network is in the Wales region, the spatial dependence model (Equation 4) is applied - using parameter values from Table 2 - to obtain:

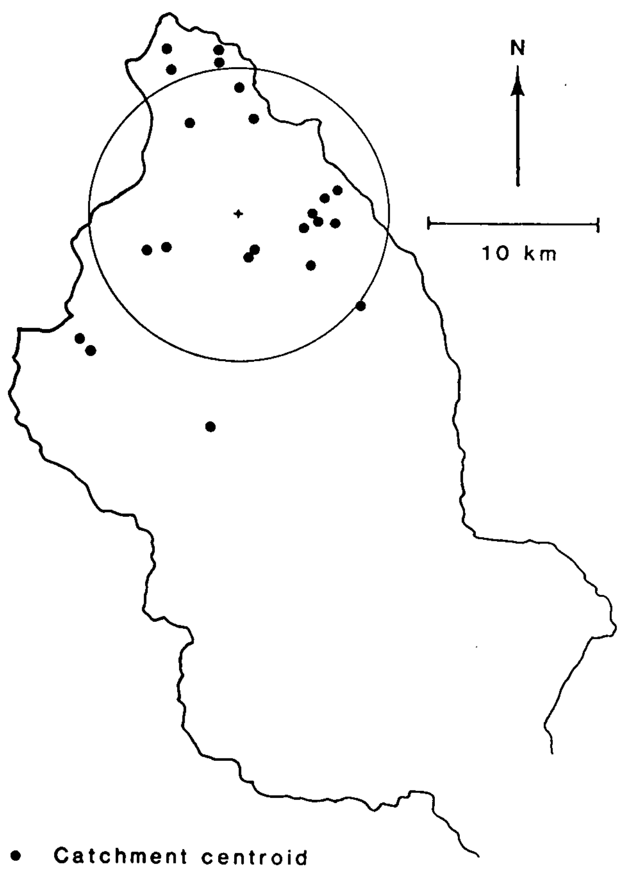
$$\begin{aligned} \ln N_e &= \ln 22\ (0.097 + 0.085\ \ln 249 - 0.052\ \ln 22 - 0.035\ \ln 0.5) \\ &= 0.430\ \ln 22\ . \end{aligned}$$

Thus $N_e = 22^{0.430} = 3.77$. It is therefore estimated that for the purpose of the collective risk assessment the 22 sites are equivalent to only 3.77 independent sites.

STEP 6: From Equation 5 it is estimated that the collective annual risk of exceedance of a 10,000-year event is:

$$\begin{aligned} r &= 1 - (1 - 1/10000)^{3.77} \\ &= 1 - 0.9999^{3.77} \\ &= 1 - 0.99962 \\ &= 0.00038 \text{ or } 1 \text{ in } 2630 \text{ years.} \end{aligned}$$

Figure 2
Impounding reservoirs in Upper Taff



B: CANAL-FEED RESERVOIRS IN THE TRENT & MERSEY

A second example of the collective risk assessment procedure is provided by considering the network of ten canal-feed reservoirs in the Trent & Mersey area, constructed in the mid-19th century. The reservoirs span an area of 3,000 km^2.

The network straddles the boundary between the Central and North West regions. Applying the Central region parameters (Table 2), and taking D=0.5 days we obtain an estimate of N_e=4.57, whereas use of the North West region parameters yields N_e=5.31. Adopting an average value of 4.94, the annual collective risk of one or more exceedances of a 1,000-year event is evaluated as:

$$\begin{aligned} r &= 1 - (1 - 1/1000)^{4.94} \\ &= 0.00493 \text{ or } 1 \text{ in } 203 \text{ years.} \end{aligned}$$

The likelihood, ℓ, of such an occurrence within a 140-year period can be calculated from:

$$\begin{aligned} \ell &= 1 - (1 - 0.00493)^{140} \\ &= 0.499 . \end{aligned}$$

Thus there is an even chance that at least one of the ten dams has experienced a 1,000-year event within its 140-year history. Of course this is only a statistical estimate; whether any of these particular dams has experienced a 1,000-year event was not researched.

C: MAJOR IMPOUNDING RESERVOIRS IN THE U.K.

The regionalization of the spatial dependence model evident in Table 2 is not so strong as to preclude application of the collective risk assessment procedure at national scale. Suppose that there are 1,000 major impounding reservoirs in the UK for which occurrence of a 10,000-year flood would provide a severe test of spillway facilities. What is the annual collective risk of such an occurrence?

An assessment of the risk can be obtained by applying the average UK spatial dependence model whose parameters are given at the foot of Table 2. Assuming an area spanned of 250,000 km^2 and a duration of 0.5 days (as before), we obtain:

$$\begin{aligned} \ln N_e &= \ln 1000\,(0.081 + 0.085 \ln 250000 - 0.051 \ln 1000 - 0.027 \ln 0.5) \\ \ln N_e &= 0.804 \ln 1000 \end{aligned}$$

Thus $N_e = 1000^{0.804} = 258$.

Hence: $r = 1 - (1 - 1/10000)^{258}$

$= 0.0255$ or 1 in 39 years.

It is not expected that this is a very reliable estimate of the collective risk of such an event. The estimate is based on extrapolation of the spatial dependence model to a much larger region than those used in its calibration. It is likely that 250,000 km^2 (which corresponds to the land area of Great Britain) is too large a spanning area, given that many of the reservoirs are clustered. However, it would seem reasonable to conclude that the annual collective risk of exceedance of the 10,000-year event at one or more of the 1,000 most significant impounding reservoirs in the UK is of the order of 1 in 40 rather than the 1 in 10 risk indicated by simple application of Equation 1.

4. Corollary to spatial dependence

The research project on which this report is based has shown that the risk of a design exceedance occurring at one or more of a network of sites is generally considerably less than that calculated if spatial dependence in heavy rainfall is neglected. An important corollary is that the risk of clustered exceedances (ie. exceedances at two or more sites occurring in a single event) is correspondingly greater than in the independent case. It can be shown (Dales and Reed, 1988) that, for design return periods of 100 years and upwards, the expected number of exceedances in years with exceedances is approximately N/N_e. This ratio provides a simple index to the degree of clustering of exceedances induced by spatial dependence.

The strong spatial dependence in heavy rainfall for the network of 22 sites considered in Example A has the implication that, when an exceedance occurs at one or more of the 22 sites, it is likely to effect several sites, since the clustering index is 22/3.77 = 5.8 . Thus, while the risk of a design exceedance occurring at one or more of the sites is about six times less (than for independence), this is offset by a corresponding expected multiplicity in exceedances.

The clustering phenomenon has important implications for the perception of collective risk. Where installations are closely grouped, design exceedances can be expected to cluster. The phenomenon is evident in the pattern of overflow incidents in storm sewer networks but, because design return periods are very much greater, is possibly not recognized with respect to networks of impounding reservoirs.

5. Application to determine operational standard of sewer network

The spatial dependence model can be applied in a different fashion to determine the typical (ie. single site) design standard underlying a given pattern of overflow incidents in a storm sewer network.

Suppose that a total of INCIDS incidents are reported in a period of M years for a sewer network with N overflow sites. In some storms, incidents will occur at several sites in the network. Thus the count (INCIDS) of incidents will be greater than the count (STORMS) of discrete storms giving rise to overflow incidents.

Given a sufficiently long period of record, an estimate of the typical operational standard of the network can be obtained from:

$$T = \frac{M \cdot N}{INCIDS} \qquad [6]$$

where T is the typical return period of incidents at single sites within the network. Such assessments are likely to be required in the aftermath of widespread flooding incidents from a particularly severe storm; moreover, it is unlikely that a long-term record of overflow incidents will be available for the current state of the network. In such circumstances the value of INCIDS will be dominated by the large number of incidents recorded in the recent event and application of Equation 6 will almost certainly lead to underestimation.

An alternative assessment of the typical operational standard is given by:

$$T = \frac{M \cdot N_e}{STORMS} \qquad [7]$$

Here, N_e is the equivalent number of independent sites obtained using Steps 1 to 5 of the collective risk assessment procedure. The assessment is not unduly sensitive to the choice of storm duration and a value of D=0.05 days is suggested for sewer network applications.

Of course, if overflows are known to be much less frequent at some sites than others, an assessment of the operational standard of the overall network may be inappropriate. However, an assessment can instead be obtained for a selected subset of overflow sites that share common characteristics, eg. those which affect residential property. A technique which may be helpful in certain circumstances is to compare the frequency of incidents at a problem site with the typical frequency of incidents at other sites in the network. The latter will be assessed more realistically by Equation 7 than using the simpler approach of Equation 6.

6. Summary

A procedure has been outlined for assessing the collective risk of a design exceedance at one of a network of sites. Examples given include one to assess the collective risk of exceedance of a 10,000-year event for a network of 22 impounding reservoirs in South Wales.

Collective risk assessments are relevant to risk management and performance monitoring rather than to "single site" flood design. Although outside the normal scope of Flood Studies Report material, it is thought that the technique may be of interest to subscribers to the Flood Studies

Supplementary Report series. A full description of the research on which the procedure is based is given in Dales and Reed, 1988.

In a wider context it is hoped that, by taking explicit account of spatial dependence, it will be possible to develop improved techniques for "pooling" data in the derivation of regional rainfall (and flood) growth curves.

Acknowledgement

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Reference

Dales M Y and Reed D W (1988) 'Regional flood and storm hazard assessment', Institute of Hydrology Report No. 102.