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The effects of urbanisation on flood magnitude and frequency

by

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ABSTRACT

This report considers the effect of urbanisation on flood hydrograph shape and on flood magnitude-frequency distribution. Urbanisation causes an increase in volume of runoff and a more rapid response - yielding a flood hydrograph that is faster to peak, faster to recede, and of increased peak discharge. The dependence of these changes on a range of factors is considered, notably the severity of the storm, the original rural response, the rainfall regime, and the location of urban area. The need is recognized for models of a range of complexity to predict urbanisation effects, and problems in model development are discussed. An extended review of literature on estimating urbanisation effects follows, in which studies have been classified as flood frequency methods, or rainfall-runoff modelling methods (including the Rational Method, the unit hydrograph method, and simulation methods). The development and analysis of flood frequency and unit hydrograph methods for particular application to UK catchments is described. Finally, initial development of a subcatchment approach to estimating the effects of urbanisation is presented.

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1. INTRODUCTION

The effects of urbanisation on catchment hydrology have been recognised for some time: changes in surface runoff, groundwater runoff, groundwater levels, and in water quality. A good account of the full hydrological consequences of progressive urbanisation is given by Savini and Kammera (1961). This report is only concerned with the effect of urbanisation on the flood flow regime, and in particular with the effect on rainfall-runoff response and on the flood frequency distribution. Furthermore, this report is confined mainly to UK conditions and the problems of arid, tropical, or cold climates are to a large extent ignored.

1.1 The effect of urbanisation on flood regime

The rainfall-runoff response of a catchment can be radically altered as a consequence of urbanisation. The introduction of "impervious" surfaces (concrete, tarmac, tile) inhibits infiltration and reduces surface retention. Thus, the proportion of storm rainfall that goes to surface runoff is increased and the proportion that goes to evapotranspiration, groundwater recharge and baseflow is reduced. This increase in surface runoff is combined with an increase in the speed of response. During urbanisation the existing drainage channels are removed or improved, and new artificial drainage is installed. The relatively slow processes of interflow in the upper soil levels and surface runoff in small rivulets leading to natural watercourses are replaced by rapid overland flow over smooth impervious surfaces, leading to drainage inlets and thence by pipe to improved drainage channels. The net result of an increase in the amount of surface runoff and a more rapid response is to produce more runoff in less time, yielding a hydrograph that is faster to peak, faster to recede and of increased peak discharge.

The magnitude of these effects, however, is not constant for a particular catchment, depending only on the degree of urbanisation, but varies from storm to storm depending on rainfall characteristics (duration, profile, relative severity) and antecedent conditions. After urbanisation, the catchment responds faster and yields runoff from smaller events. It is therefore able to respond more fully to shorter bursts of rainfall, of smaller depth but greater intensity. Consequently, peak discharges following such short, intense storms tend to be increased in greater proportion than peaks due to longer storms of more uniform intensity. This effect, however, may be offset to some extent by the severity of the event. Severe storms, having intensities much greater than the infiltration rate (either because the rainfall is very intense, or because the catchment is already wet and of reduced infiltration capacity), would have yielded response from the rural catchment that already resembled the urban catchment - high percentage runoff and rapid overland flow. Thus peak discharges due to severe storms may not be increased by as large a proportion as peaks due to relatively frequent, less severe storms. A further consideration, however, is the effect of the urban drainage system. Severe storms may yield discharges exceeding the capacity of the sewer system, causing choking of the flow and increased attentuation in localised ponding. Further downstream, however, urban encroachment onto the flood plain and levée construction reduce the available overbank storage and reduce also the catchment's natural ability to attenuate overbankfull discharges. The net effect of urbanisation on peak discharges due to severe storms is thus catchment specific, but in general smaller increases may be expected than for less severe storms.

The effects of urbanisation have been discussed above in terms of catchment response. These effects, however, yield corresponding changes in the statistical distribution of floods. Mean annual flood is increased, but because the effect on severe storms is generally not so great, rarer floods are less increased. Consequently, the coefficient of variation of the distribution is reduced, and the slope of the growth curve (the graph of the ratio "T-year flood to mean annual flood" against T) is reduced. Also, because of the reduced importance of antecedent conditions and the increased sensitivity to short duration, high intensity rainfall events (which in the UK become more frequent with the increase in convective rainfall in the summer) there is a progressive tendency for the flood season to move from winter to summer.

1.2 Variation in the effect of urbanisation between catchments

Section 1.1 has considered the effect urbanisation has on the flood regime of a catchment: increased percentage runoff and a more rapid response yielding increases in peak discharges and a more "flashy" flood hydrograph, and correspondingly an increase in mean annual flood, but a possible reduction in the slope of the flood frequency growth curve. These effects have been recognised for some while, but accurate estimation of the magnitude of the effects for particular catchments has proved consistently difficult, and many studies have yielded inconsistent or inconclusive results. A summary of a large number of studies (taken from Riordan *et al.* 1978) is given in the Appendix 1. This shows a range in the reported effects of urbanisation on (i) percentage runoff varying from no effect to a six-fold increase, on (ii) response time varying from no effect to a 10-fold decrease, and on (iii) mean annual flood varying from no effect to a 10-fold increase. Part of this range in results may be due to random and systematic errors and differences between the studies, however, the huge range suggests more fundamental differences. In this respect the following considerations may be of particular significance:

(i) the original rural response

In assessing the effect of urbanisation, it is the change in catchment response relative to some original response that is sought. This original response is therefore fundamental to the problem. By representing the effect of urbanisation as a simple factor related only to the extent of urbanisation no allowance is made for variation in rural response between catchments. The primary effects of urbanisation, an increase in percentage runoff and an increase in rapidity of response, will obviously be more significant if the original rural response consisted of low percentage runoff and a sluggish response than if it already consisted of high percentage runoff and a rapid response. Moreover, the net effect of urbanisation will depend on the relative magnitude of the change in percentage runoff compared with the change in rapidity of response. For catchments which already yield extensive surface runoff the increase in percentage runoff may be barely significant, and the main effect of urbanisation would be the change in response time. Alternatively, for catchments which previously gave only small amounts of surface runoff, the change in percentage runoff might be more significant than the change in rapidity of response. To assess the effect of urbanisation successfully, increases in percentage runoff and rapidity of response should be considered separately.

(ii) The local rainfall characteristics

A repercussion of the increased percentage runoff and more rapid response is that, as discussed earlier, the T-year flood after urbanisation tends to be caused by a shorter more intense storm than before. Consequently the effect of urbanisation will vary to some extent with local rainfall characteristics, and in particular with the relationship between typical rainfall intensities for short and long duration storms. Moreover, since the T-year flood after urbanisation tends to be caused by a shorter storm, the flood frequency growth curve will tend more towards the rainfall growth curve for shorter durations than longer durations; this will affect the expected reduction in flood frequency growth rate, and could even cause an increase.

(iii) The relationship between impervious area and the drainage network, and the location of urban development within the catchment.

From the description given in Section 1.1 it might be assumed that the increases in percentage runoff and rapidity of response arise from separate causes, the increase in percentage runoff due solely to the increase in impervious area and the increase in rapidity of response due solely to the improved drainage system. While this may be true to a first approximation, the change in each of percentage runoff and rapidity of response will depend also on the interaction between the impervious area and the drainage system, and on the location of the urban development within the catchment. The effect on percentage runoff is considered first.

On natural catchments surface runoff occurs in two instances (i) when the incident rainfall intensity is greater than the infiltration rate and (ii) when the interflow discharge exceeds the capacity of the upper soil layers (which become 'wet' and of zero infiltration capacity) and is forced to emerge as surface flow. During rainfall, the interflow discharge increases causing the 'wet' areas adjacent to the drainage system to expand into the more remote dryer areas. After the rainfall has ceased, the 'wet' areas will drain and contract. Catchment runoff can then be considered as the joint effect of (i) the emergent interflow, (ii) approximately 100% runoff from the 'wet' areas, (iii) any rainfall excess from the damp areas around the 'wet' areas and (iv) approximately zero runoff from the 'dry' areas beyond. In this context, the location of impervious surfaces, and whether or not they are directly connected to the drainage system may have a significant effect on percentage runoff.

Although percentage runoff may be increased by the introduction of impervious surfaces, if the surfaces are located in areas that were previously "wet" (in the valley floor or flood plain) the change in percentage runoff may not be so great as if the impervious areas are located in previously "dry" areas. Moreover, if impervious surfaces are introduced without improvements to the drainage system, the effect on percentage runoff may be slight - impervious area runoff soaking away on adjacent pervious surfaces. In this respect, the introduction of kerbed roadways may be of particular significance.

Percentage runoff may also be increased by the improvements to the drainage system alone. Whereas the natural drainage system might vary in length and density with season and soil conditions, the artificial drainage system is more constant and may reach areas, pervious and impervious, that previously did not contribute to storm runoff. Since this effect would be more evident during drier seasons it would contribute to a change in the seasonal distribution of flood flows.

As stated earlier, the interaction of impervious area with the drainage system and the location of urban development within the catchment affect rapidity of response as well as percentage runoff. The increase in rapidity of response arises from increased velocity of both channel and surface flow. In large catchments, channel flow represents the major proportion of total flow time and thus the increase in rapidity of response may be closely related to the improvements made to the drainage system. However for small catchments, surface flow represents a significant proportion of total flow time. Thus the introduction of impervious surfaces, by increasing the velocity of surface flow, may alone represent a significant proportion of the increase in rapidity of response. Furthermore, part of the increased rapidity of response may be due to a change in the time distribution of rainfall losses. Runoff from the new impervious surfaces may begin almost immediately whereas on the original pervious surfaces some rainfall was lost as infiltration before runoff could begin. Thus an increase in impervious area can lead to a more uniform distribution of rainfall loss over the duration of the storm and so contribute to a more rapid response.

The location of urban development affects not only the percentage runoff from different parts of the catchment but also the relative flow times and phasing of response. If urban development has taken place at a point remote from the catchment outfall, the quicker urban response may arrive at the outfall at the same time as the slower response from the rural areas nearer the outfall, thus yielding a reinforced peak discharge. Conversely, if urban development has taken place near the outfall, the quicker urban response may have passed the outfall before the rural response from the more remote areas has arrived, thus yielding a lower or even double peaked hydrograph. Note, however, that although such downstream urban development may not cause concern at that design point (the outfall), it may cause peak reinforcement at points further downstream.

The above discussion has tried to identify the main reasons why the effect of urbanisation is seen to vary between catchments. Several other less tangible reasons may also be significant. Two such reasons were mentioned in Section 1.1; the effect of localised ponding caused by the flow exceeding the storm drainage capacity, and the effect of flood plain development. Several other reasons are considered below.

Firstly, percentage runoff may be increased by building activity itself. Construction processes disturb the natural top soil. With recompaction, reinstatement, and turfing the infiltration capacity of the upper soil layers may be much reduced. Also, interflow paths may be blocked or destroyed, causing any interflow to "well up" onto adjacent impervious areas, and thence into the artificial drainage system. However, the establishment of domestic gardens may have the reverse effect, breaking up the top soil and increasing local infiltration rates.

Secondly, the introduction of impervious surfaces and the improvements to the drainage system may cause a general lowering of groundwater levels and hence a reduction in catchment wetness. However the introduction of a water supply and effluent disposal system may have the opposite effect, with distribution losses and garden and municipal irrigation artificially increasing catchment wetness and also therefore increasing percentage runoff.

Thirdly, the provision of planned local storage and soakaways in some areas has alleviated and virtually eliminated the expected effect of urbanisation.

Fourthly, lack of maintenance of urban sewer systems and open channels can lead to increased localised ponding, thus reducing flows downstream but also resulting in local discomfort.

Finally, the effects of urbanisation discussed above concern only changes in the flood runoff processes and it is usually assumed that the rainfall process is not affected by urban development. There is however growing evidence of local climate change with urbanisation (WMO, 1970). The formation of an urban "heat island" together with increased atmospheric turbulence and an increased abundance of condensation nuclei can lead to significant increases in rainfall. Such increases are difficult to estimate since they depend on size and type of urbanisation and on the existing local climate. Increases of 5 to 10% have been estimated from rainfall records in large cities, but such increases are usually neglected in estimating flood magnitudes since, as large cities take many years to develop, any early effects will already be incorporated in the local depth-duration-frequency data.

2. ESTIMATING THE EFFECTS OF URBANISATION

2.1 The need for a method to estimate the effects of urbanisation

The main requirement of hydrological design in urbanising catchments is for models capable of spanning the progression from natural to urban conditions, and capable of providing design flood information at any stage of urbanisation. Traditionally a gulf has existed between methods of design flood estimation for rural and urban catchments. Flood estimation for rural catchments has evolved along two lines: the statistical flood frequency approach, based on long records of observed floods flows; and the deterministic rainfall-runoff modelling approach based on simulating the observed response of the catchment to rainfall. The two approaches may be considered complementary in that the first provides an estimate of flood level at a specified probability while the second provides an estimate of both flood level and hydrograph shape due to a design storm of specified probability (though the probabilities of derived hydrograph and design storm have seldom been related). With these two approaches, it has been possible to decide whether the better alternative for flood control is to contain the flows by channel improvement and levée construction or reduce the flows by reservoir storage and attenuation.

By contrast, flood estimation for urban catchments has evolved almost exclusively along the deterministic rainfall-runoff modelling approach; the urban hydrological system would appear to lend itself well to the deterministic approach since firstly the "contributing area" of the catchment is relatively constant (being closely related to the impervious area), and secondly the drainage system is well defined (consisting predominantly of flow over plane surfaces and in gutters, pipes and channels). Urban flood estimates can, with reasonable confidence therefore, be based almost entirely on hydraulic principles, making only limited use of observed rainfall and runoff data. The virtual absence of the flood frequency approach for urban catchments can be directly attributed to this apparent lack of need for observed data; long records of annual maximum floods from catchments in a fixed state of urban development are not generally available. Urban flood frequency therefore is usually estimated using deterministic models, assuming the resultant flood peak has the same probability as the rainfall depth in some critical storm duration. It is becoming increasingly apparent, however, that urban rainfall-runoff models generally yield poor estimates of flood frequency, the combined effect of "factors of safety" often leading to considerable overestimation. More reliable techniques of estimating flood frequency are required, particularly when the design level of flood protection is to be determined by economic analysis.

Another feature of urban drainage design is that surface water in an urban environment has tended to be considered a nuisance to be conveyed swiftly from the catchment by an efficient drainage system. Thus traditional urban drainage design methods give an estimate only of peak flow, which the drainage system is then designed to pass. More recently, with rapid urbanisation and New Town Development, it has been realised that passing on the increased flood waters simply compounds the problem downstream, and thus may not represent the optimal solution on economic or environmental grounds. Planning Authorities now tend to specify that development should not alter the existing flood-frequency distribution beyond prescribed limits. The concept of Blue-Green development has evolved, whereby small flood control reservoirs and temporary storage areas, designed to both 'balance' the increased flood potential due to urbanisation and to replace some of the overbank storage lost to flood plain development, are sited in 'linear parks' of designated public open space or parkland along the natural water-courses. Thus not only is urban flooding alleviated but also the urban environment is enhanced. For the design of such flood alleviation works a complete design hydrograph is a vital prerequisite.

The need for a model to span the gulf between urban and rural conditions exists at three levels. Firstly, for planning, a simple model relating flood levels to urban development is needed to allow the evaluation of alternative development schemes. Such a model should be 'desk-top' or interactive computer based needing as input only the very basic data available at the planning stage, but also, if possible, able to account broadly for the effects of location of urban development and flood control structures. Secondly, for major drainage design, a more detailed model is required to simulate accurately the response of the urbanising catchment to enable the detailed design requirements of flood alleviation works to be specified. Such a model will have to be able to account explicitly for the effects of location of urban development and for flood control structures. The data requirements and computational complexity will almost certainly necessitate a computer based model. Thirdly, for local drainage design, a sewer design model is required. Such a model would logically provide the input for major drainage design, but in practice, these models are generally too complex or too detailed for extension to larger urban catchments. Furthermore, local drainage is often not designed until after the major drainage works have been completed, since detailed information on pipe layout and land gradient may not be available until late in the development and may be subject to continual alteration. Some sewer design models are capable of simplification for use at the planning stage, and a recent report by Price *et al.* (1980) describes the development of such a model. A review of sewer design models can be found in Colyer and Pethick (1976). The present report is concerned with planning models and models for major drainage design downstream of the local sewer system.

2.2 Approaches to modelling urbanising catchments and problems involved

There are two basic approaches that may be adopted in order to identify the effects of urbanisation. In the first, a catchment is monitored throughout an urbanising period (with possibly a nearby rural catchment being monitored as a control). Thus information is obtained directly on the change in rainfall-runoff response, but usually, because of the shortness and non-stationarity of the record, no information on the change in flood frequency is available. Moreover, general trends can only be identified when the results from many such studies are collated. In the second type of study, the rainfall-runoff and flood-frequency characteristics of several catchments, both urban and rural, from within a region are compared, and any differences related to catchment characteristics including the degree of urbanisation. Results, however, need careful interpretation to ensure that urbanisation effects have been adequately separated from those of other catchment characteristics. In either type of study the model builder is generally faced with 4 main problems:

(i) An increase in the complexity of the hydrological cycle

The magnitude of any changes in the rainfall-runoff processes will depend on: catchment size, slope and soil characteristics; the pattern of development; the interaction between pervious areas, impervious areas and the drainage system; and variation in the distribution of available flood storage. Changes in the flood frequency distribution will depend also on the changes in flood season. McPherson and Schneider (1974) give a full summary of many of the problems involved.

(ii) The problems of parametric description of urban development

Percentage urban area or percentage paved area are readily defined from maps or areal photographs, but accurate estimation can be quite time consuming. Many workers consider typical percentage imperviousness for certain development types and derive a weighted average over the catchment. Others have related percentage imperviousness to population density (eg Stankowski, 1972), or used satellite radiation scans to identify land use types (eg Ragan, 1975, use of LANDSAT data). The choice of parameters to define channel improvement and the distribution of urban development is less obvious. Furthermore, since drainage improvements and increases in paved area tend to occur simultaneously, parameters describing these changes tend to exhibit high statistical inter-correlation. The use of such parameters in prediction equations thus requires care, and the estimation of changes due to each parameter individually, or the extension of such equations to new catchments with different configurations of development may not be valid.

(iii) A general lack of reliable data

In order to identify the effect of urbanisation among other effects due to, for example, variation in rainfall or antecedent conditions, data of high quality are required. To model the effect of urbanisation on flood frequency, long records from catchments in stable condition of urbanisation are required, whereas to model the effect on the rainfall-runoff process good synchronised rainfall-runoff records at short time intervals are needed.

(iv) Difficulty in generalising the results

To be able to apply the model to estimating the effects of urbanisation in an ungauged catchment (and future conditions in a gauged catchment are always ungauged), it must be possible to estimate model parameters from catchment characteristics. The more empirical methods (eg. flood-frequency methods and unit hydrograph methods) are usually fitted to observed rainfall-runoff data and the chosen optimum parameters regressed on catchment characteristics. The use of such equations to estimate model parameters for conditions not truly comparable to those on which the equations were derived may lead to errors. This caution applies particularly to urbanising catchments since urbanisation may change the natural areal distribution of contributing areas and overland flow lengths, which before conformed to some regional statistical norm. In spite of these reservations, considerable experience in the use of empirical methods has been passed on, and the chance of inexperienced users obtaining gross errors in their application is generally less than for the more complicated physically based models. With these latter models, the physics of catchment response is more realistically modelled, and model parameters are intended to correspond to specific catchment characteristics (eg depth to impermeable soil layers, overland and channel flow lengths and roughnesses). It may then be assumed that the effects of urbanisation may be adequately estimated by the intuitive adjustment of model parameters. However, since the model is only a simplified representation of the catchment, this can lead to errors, and the assumption should be tested by fitting the model to observed pre- and post-urbanisation records. Ideally model parameters should be regressed on catchment characteristics, but because of the cost of fitting the models to a large number of catchments (both in terms of time and money) and because of the problems of estimating a large number of model parameters accurately, few attempts have been made, and even fewer have been successful.

3. REVIEW OF EXISTING DESIGN METHODS & RELATED RESEARCH

3.1 Introduction

Effective design of river management works for flood alleviation in urban and urbanising catchments has long been a problem for the design engineer. Faced with the huge variability of nature, he has used engineering judgement tempered by past experience. To this end several formulae and methodologies have been developed, usually involving the estimation of coefficients within fairly well defined ranges. However, many hydrologists believed the mechanics of flooding could not be so easily generalised, and Wellington's (1886) comment on an early formula for culvert design was typical.

"It is natural for fallible man to wish to reduce everything to rule, even if it be only rule of thumb. The responsibility of the individual is much diminished if he has something of that kind to lean on and in the proper sizing of culverts this is especially natural. It is well, however, to be certain we are not simply making a rule where there is no rule, and so laying the foundation for future trouble".

In spite of such reservations, the use of "rules" for determining design flows has become standard practice, both for the design itself and for demonstrating the adequacy of the design to planners and politicians. However, Wellington's reservations are still valid today, and any rule for predicting the effect of urbanisation should be viewed with a healthy scepticism. In particular, methods should be soundly based and proved on data from a wide variety of catchments.

As mentioned in previous sections, two basic approaches to flood estimation have evolved: the statistical flood-frequency approach, based on fitting theoretical or empirical distributions to long records of flood data, and the deterministic rainfall-runoff modelling approach based on estimating the runoff pattern from individual rainfall events. The two approaches originally evolved almost completely independently and their application to predicting the effect of urbanisation will be traced separately. However, it is now becoming common for flood frequency models and rainfall-runoff models to be used in conjunction: flood frequency analysis being applied to the output from the rainfall-runoff model. Such hybrid approaches are difficult to classify (as one or the other), but an attempt has been made to do so in this report depending on what the main purpose of the study was considered to be.

The form of presentation adopted for this review is, within each of the classified subject areas, to provide a general introduction to the basic concepts, followed by what is essentially an annotated list of studies of the effects of urbanisation in chronological order. Some attempt has been made to show linkages between the various studies. Those studies which the author considers particularly important are marked by an asterisk in the margin. The reader may prefer at first to consider only those "starred" sections.

3.2 Flood-Frequency Techniques

A. General introduction

Flood frequency techniques are based on considering observed series of flow quantiles (annual peaks, volumes, peaks over thresholds) as samples from an underlying probability distribution. As long as successive quantiles in the sample are independent (uncorrelated), and as long as the underlying distribution does not change during the sample period (ie no trend towards - for example - increasing flood peaks with time is present) the sample may be analysed by standard statistical methods to yield estimates of the underlying distribution. The underlying distribution may then be used to estimate the flood of any required probability (the probability, P, of a flood being exceeded in any year is usually expressed as an average return period, $T = 1/P$).

Two approaches to estimating the underlying distribution may be applied, the theoretical (parametric) and the empirical (observed). In the parametric approach, an assumption as to the form of the underlying distribution is made, and the parameters of that distribution estimated from the statistics of the sample (mean, median, quartiles, standard deviation, skewness, etc ...). In the empirical approach, the form of the underlying distribution is not assumed *a priori*, but is defined from the sample. The observed series (of number N say) is ranked, and observed probabilities assigned to each flow such that the largest in the sample has a probability of approximately 1/N of being equalled or exceeded, the second largest a probability of approximately 2/N of being equalled or exceeded, and so on. Full details of analytical procedures for both parametric and empirical approaches may be found elsewhere (eg Chow, 1965; NERC, 1975).

Flood-frequency techniques were first used towards the end of the last century and were being further developed at the beginning of this century. Horton (1913) presented applications of the normal distribution, and Fuller (1914) presented a comprehensive study of flood-frequency methods including the use of frequency factors (or growth factors) to define the T-year flood in terms of the mean annual flood. Recognising the skewed nature of observed flood frequency distributions, Hazen (1914) proposed the use of the Log-Normal distribution, and Foster (1924) proposed the use of the Pearson type I and III distributions to allow the exact fitting of skewness. Several other theoretical and empirical distributions have been proposed, but it was Gumbel's (1941) presentation of Fisher and Tippet Extreme Value Theory: the type I (Gumbel); type II (Frechet); and type III (Weibull) distributions, that first provided theoretical backing for particular distributions and established flood-frequency analysis as a valid design tool.

Two major drawbacks with flood-frequency analysis have been the difficulty of transferring results to ungauged sites and the unreliability of estimates of the underlying distribution based on small samples. To overcome these problems, regional correlation analysis has been used; Kinnison and Colby (1945) related peak flows to catchment characteristics, and Dalrymple (1950) proposed regional analysis. Following these techniques, the U.S. Geological Survey (USGS) initiated a series of regional analyses (see Dalrymple, 1960) in which (i) mean annual flood was related to catchment characteristics, and (ii) individual station records were combined to define, empirically, a single regional average growth curve of frequency factor (T-year flood to mean annual flood) against T. The US Army Corps of Engineers have adopted a different approach (see Beard, 1962) and recommend fitting the theoretical Log-Pearson III distribution to individual station records using a regionalised value for skewness. More recently, Benson (1968), reporting on the findings of a working group on flow frequency methods, has also recommended the Log-Pearson III distribution as an objective and "uniform" method for U.S. federal agencies. Having fitted the distribution, floods of specific return periods (usually 2, 5, 10, 25, 50 and 100 years) may be estimated and related to catchment characteristics. In this way, although an assumption on the form of the distribution is made, the assumption that the $Q/\bar{Q}$ against T relationship is constant within some arbitrarily defined region is avoided. In the UK Flood Studies Report (NERC, 1975), however, Dalrymple's regional analysis approach was adopted since many of the available records were short (less than 10 years) and estimation of even 10 year floods from individual stations was subject to large errors.

B. The effects of urbanisation

One of the first flood frequency studies to make any recommendation on the effects of urbanisation was Bigwood and Thomas (1955). They recommended (i) increasing mean annual flood estimates by about 2 and 3 times respectively for suburban and urban residential development, and (ii) using the same growth curve for rural and urban areas.
* Carter (1961) however is usually credited with the first fundamental study of the effects of urbanisation. Reasoning that any change in flood magnitudes was due both to an increase in percentage runoff and to a more rapid response, he used data from 18 catchments in the Washington

area to obtain an equation for mean annual flood in terms of basin imperviousness, lag time and area. Lag time was defined as the time from centroid of effective rainfall to the centroid of direct runoff. For use in ungauged catchments several procedures for estimating lag time are available (see also section 3.3.4 on rainfall-runoff modelling and summary of lag time equations in Appendix 3). Carter presented a graphical relationship for lag time based on the basin ratio (Stream Length/√Stream Slope) and the degree of sewerage. He found suburban development reduced lag times by 60% and increased mean annual flood by 80%; fully developed catchments however had lag times reduced by 80% and mean annual flood increased by 200%. The effect of urbanisation on flood frequency growth curves was not considered.

Carter's work and the continued USGS regional analysis programme stimulated further research. Wilson (1967), after completing a USGS regional flood-frequency analysis in Mississippi, went on to analyse 4 urban streams in Jackson. He presented an average growth curve which was significantly flatter than his earlier rural growth curve. He also related mean annual flood to basin area and the percentage of the area sewered. The work suggested complete urbanisation increased mean annual flood by 350% but increased the 50 year flood by only 200%.

* Anderson (1968, 1970) built on Carter's (1961) analysis of mean annual flood, extending it to include 44 catchments in all from the Washington-Virginia area. Furthermore, he proposed a method of predicting the effect of urbanisation on both mean annual and rarer floods. Anderson's equation for mean annual flood is given as:

$$\bar{Q} = 230\ (1 + .015I)\ A^{.82}\ T_L^{-.48} \tag{3.1}$$

where $\bar{Q}$ is mean annual flood (ft^3/s)

I is percentage of catchment impervious

A is catchment area (mi^2)

T_L is lag time (hr)

The factor $(1 + .015I)$ is meant to represent the factorial increase in percentage runoff from 30% at $I = 0\%$ to 75% at $I = 100\%$. For the ungauged situation, lag time is defined graphically (see Fig. 3.1) in terms of the degree of drainage improvement and the basin ratio, $L/\sqrt{S}$ (where L is distance (mi) along the main channel from outlet to divide and S is slope (ft/mi) of main channel from .1L to .85L upstream of outlet). Figure 3.1 is based on data from 79 catchments. The T-year flood (Q_T) is estimated from the mean annual flood using the "urbanised" flood frequency growth curve. This growth curve was determined by interpolation, using the factor $(1 + .015I)$, between the rural growth curve (found by regional analysis) and the 100% impervious growth curve (assumed equal to the rainfall growth curve). That is:

$$Q_T = \bar{Q}.\ \frac{R_n + .01I(2.5R_{100} - R_n)}{(1 + .015I)} \tag{3.2}$$

where R_n is the T-year factor on the rural growth curve, and R_{100} is the

T-year factor on the average rainfall growth curve (averaged over a range of rainfall durations from 5 minutes to 24 hours). From these equations Anderson concluded that suburban development typical of the study area increased mean annual flood by between 100 and 200%. The 100 year flood, however, was increased by only 100 to 150%.

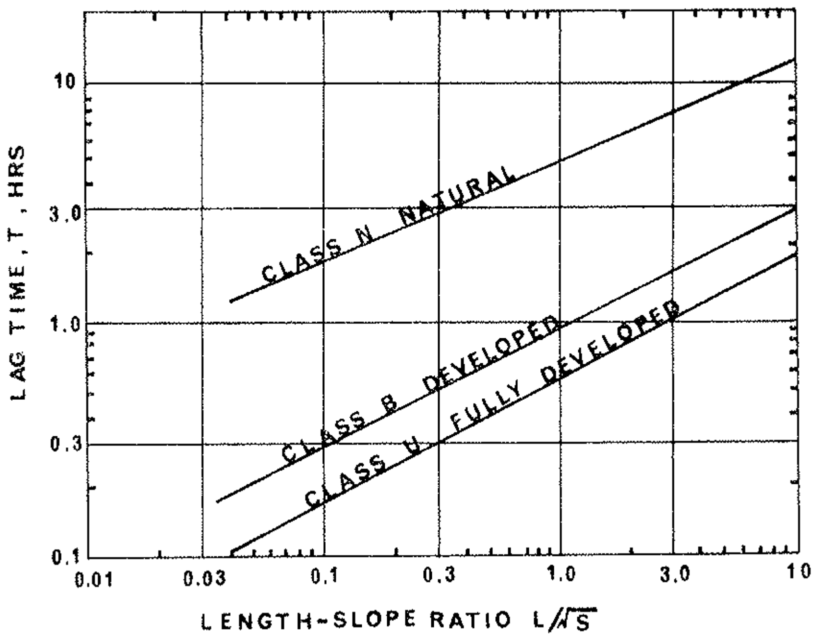


FIGURE 3.1 **Lag time against L/√S for three classes of urbanisation** *(from Anderson, 1968)*

These, or similar, procedures have been used by many workers - developing their own regional $\bar{Q}$ equation (3.1), their own lag time relationship on $L/\sqrt{S}$ and degree of sewerage (or perhaps percentage imperviousness), and their own regional curves for R_n and R_{100} (see Wiitala, 1961; Martens, 1968; Gann, 1971; Sauer, 1974; Knight, 1976; Thomas and Corley, 1977; Golden, 1977). The procedures have also been used by Robey (1970) to adjust an observed flow record to "present day" urbanisation. His approach of applying the ratio of urban to rural mean annual flood to the observed flood series, however, assumes urbanisation affects only the mean of the frequency distribution and not the slope of the frequency curve.

* Leopold (1968) summarised the results of several studies, and presented a graph (see Figure 3.2) of mean annual flood per square mile against percentage of the catchment impervious and the percentage sewered. Using this graph and the rural growth curve he sketched in, by intuition, growth curves for various degrees of imperviousness and sewerage. Several workers have used Figure 3.2 for the effect of urbanisation on mean annual flood with their own procedure for the urbanised growth curve. Gann (1971) combined Figure 3.2 with Anderson's (1968) approach for the urbanised growth curve (equation 3.2). Sauer (1974), however,

used Figure 3.2 to estimate R (the ratio of urban to rural mean annual flood) and noticing R had a maximum value of 7, he presented a slightly different method from Anderson's (1968) for estimating the T year flood:

$$Q_{T,u} = \{7\ R_{T,100}\ Q_2(R-1) + Q_T(7-R)\}/6 \qquad (3.3)$$

where $Q_{T,u}$ is the T-year flood on the urbanised catchment

Q_T is the T-year flood on the rural catchment

Q_2 is the 2-year flood on the rural catchment ($\approx\tilde{Q}$)

and $R_{T,100}$ is the T-year frequency factor for rainfall.

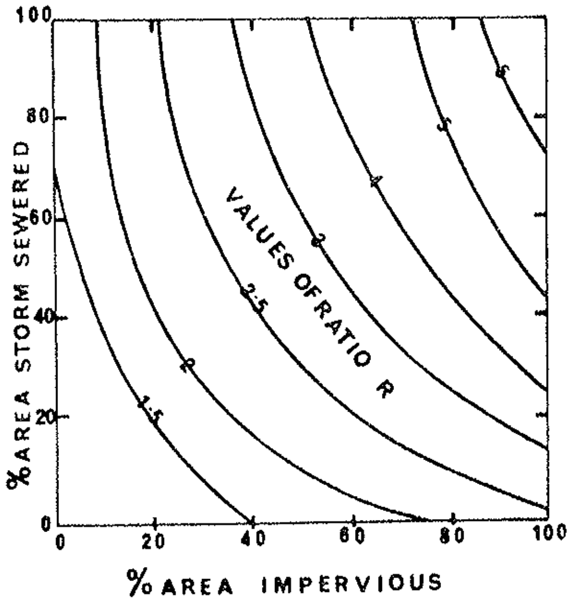


FIGURE 3.2 **Ratio R of urban to rural mean annual flood**
(from Leopold, 1968)

Sauer presents equations for the rural T-year flood (where T=2, 5, 10, 25, 50 and 100) in terms of catchment characteristics. Thomas and Corley (1977) have updated Sauer's work including more small catchments in the equations for Q_T. Data from 3 urban catchments were included yielding relationships that were in broad agreement with Sauer's procedure (equation 3.3).

Included in Leopold's (1968) summary was James' (1965) work in applying the Stanford Watershed Model to a catchment in California (see section 3.3.5). Rantz (1971) also considers James' work and presents a series of figures similar to Leopold's for each of 2, 5, 10, 25, 50 and 100 year return periods (Fig 3.3).

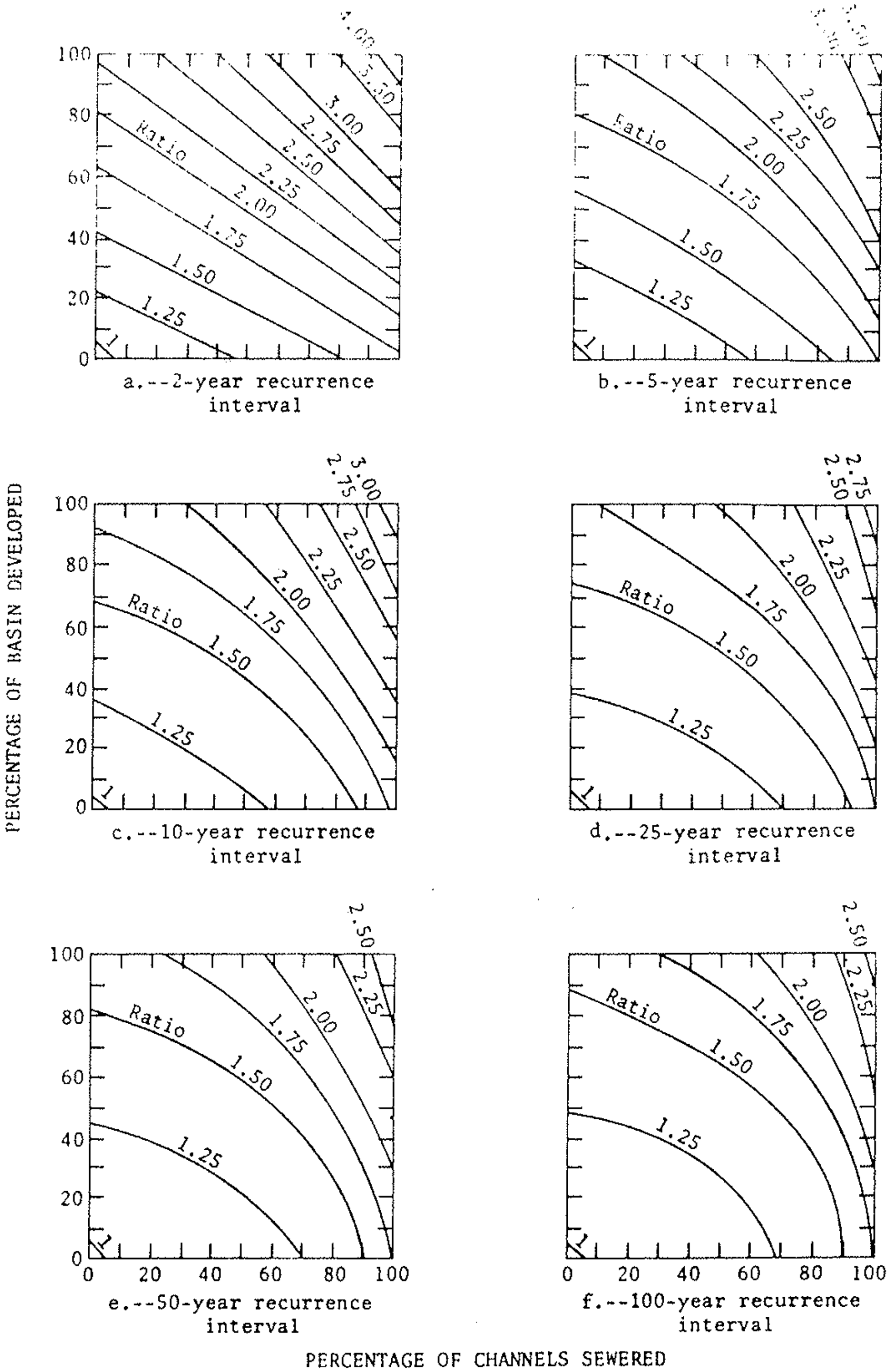


FIGURE 3.3 **Rantz' (1971) curves for the effect of urbanisation on T-year flood**
(based on work of James, 1965)

Note axes are reversed compared to Figure 3.2.

* Putnam (1972) appears to be the first to estimate separate equations for 2, 5, 10, 25, 50 and 100 year floods in catchments subject to urbanisation. He presents the following equations based on 45 catchments in North Carolina:

$$Q_2 = 221\ A^{.87}\ T_L^{-.60} \qquad Q_5 = 405\ A^{.80}\ T_L^{-.52} \qquad (3.4)$$

$$Q_{10} = 560\ A^{.76}\ T_L^{-.48} \qquad Q_{25} = 790\ A^{.71}\ T_L^{-.42}$$

$$Q_{50} = 990\ A^{.67}\ T_L^{-.37} \qquad Q_{100} = 1200\ A^{.63}\ T_L^{-.33}$$

where: Q_T is the flood peak (ft^3/s) of return period T years

A is catchment area (mi^2)

T_L is lag time (hr)

It should be noted, however, that the coefficients and exponents in the Q_{50} and Q_{100} equations were found not by regression, but by graphical extrapolation of the corresponding values in the Q_2, Q_5, Q_{10} and Q_{25} equations. For application to ungauged catchments Putnam obtained an equation for lag time, in terms of basin ratio and percentage imperviousness, I, based on 118 catchments

$$T_L = 0.49\ (L/\sqrt{S})^{.50}\ (I/100)^{-.57} \qquad (3.5)$$

Equation (3.5) is based on a range of I from 1 to 32. Substituting these values into equation (3.5) gives coefficients for $(L/\sqrt{S})^{.5}$ of 6.8 and .94 respectively which compare favourably with Anderson's graph (see Fig. 3.1), and suggest fully urbanising a catchment reduces lag time by about 85%. The corresponding effect on flood peak is to increase 2 year flood by 230% and 100 year flood by 90%. Following Putnam's work, separate equations for the effect of urbanisation on T-year flood have been derived by a number of workers, in particular; Johnson and Sayre (1973); Espey and Winslow (1974); Stankowski (1974); Dempster (1974) and Weiss (1975).

McCuen and James (1972) demonstrate the use of non-parametric statistics to identify urbanisation effects in annual maximum series. Lazaro (1976) has also used non-parametric statistics, but the approach only determines whether any increase in flood magnitude is statistically significant and does not relate the increase to catchment characteristics.

Hammer (1973), in a study concentrating on stream channel changes with urbanisation, considered the changes in flood-frequency distribution with urbanisation by(i) relating the increase in mean annual flood to the increase in population density, and (ii) relating the change in growth curve, as evidenced by the change in the fitted Gumbel parameters, to urbanisation, as evidenced by the stream channel enlargement. For application to ungauged sites, he presents a regression equation for stream channel enlargement in terms of 10 independent variables.

* Johnson and Sayre (1973) developed equations for 2, 5, 10, 25, 50 and 100 year floods based on 28 catchments from Houston, Texas. Observed rainfall-runoff data from each catchment were used in a regression analysis to relate individual flood peaks to rainfall depth, duration and antecedent moisture condition. Then, using a 60 year rainfall record, synthetic annual maxima series were derived for each catchment. Estimates of the 2, 5, 10, 25, 50 and 100 year floods were made by fitting the Log-Pearson III distribution. Equations were then derived relating flood discharges (ft^3/s) directly to catchment area (mi^2) and percentage imperviousness, without using the intermediate variable lag time (as used by Carter, Anderson and Putnam). The equations for 2 and 100 year return period are presented here:

$$Q_2 = 38.8\ A^{0.86}\ I^{0.62}$$

$$Q_{100} = 156.\ A^{0.89}\ I^{0.45} \qquad (3.6)$$

The range of I was from 1 to 35%, giving increases in the 2 and 100 year floods of 800% and 400% respectively, figures much larger than Anderson's or Putnam's. It should be borne in mind, however, that their results are based on synthetic data and this may to some extent explain the large increases.

Espey and Winslow (1974) fitted the Log Pearson III distribution to data from 60 USA East Coast and Texas catchments. They present separate equations for 2.33, 5, 10, 20 and 50 year floods using the Texas catchments alone, the East Coast catchments alone and the combined catchment set. Their results for the Texas subset (mainly Houston area) predicted increases in the 2.33 and 50 year floods for I increasing from 1% to 35% of 370% and 540% respectively. These increases are quite different from those predicted by Johnson and Sayre, and indeed show a greater effect of urbanisation for rarer floods. For the East Coast subset Espey and Winslow found no significant effect of urbanisation. Combining both sets of catchments they derived equations of the following form (only those for return period 2.33 and 50 years are presented here):

$$Q_{2.33} = 169\ A^{0.77}\ I^{0.29}\ S^{0.42}\ R_{2.33}^{1.80}\ \phi^{-1.17}$$

$$Q_{50} = 297\ A^{0.85}\ I^{0.22}\ S^{0.50}\ R_{50}^{1.57}\ \phi^{-1.61} \qquad (3.7)$$

where: Q_T is flood peak (ft^3/s) of return period T

A is catchment area (mi^2)

I is percentage imperviousness

S is slope (ft/ft)

R_T is 6-hour rainfall (in) of return period T

ϕ is channelisation factor (see Table 3.4 section 3.3.4)

These equations typically predict increases in 2.33 and 50 year floods of 410% and 400% respectively, just reversing the trend for the Texas subset alone. Channelisation factor, ϕ, makes a significant contribution to any increases, particularly for rarer floods, but its estimation is largely subjective.

Stankowski (1974) fitted the Log Pearson III distribution to 103 catchments in New Jersey, and presented equations for 2, 5, 10, 25, 50 and 100 year floods though only those for 2 and 100 year return periods are presented here:

$$Q_2 = 25.6\ A^{.89}\ S^{.25}\ S_t^{-.56}\ I^{.25}$$
$$Q_{100} = 136\ A^{.85}\ S^{.26}\ S_t^{-.51}\ I^{.14} \qquad (3.8)$$

where: Q is flood peak (ft^3/s)

A is catchment area (mi^2)

S is main channel slope (ft/mi)

S_t is per cent of area under lake or swamp

I is percentage imperviousness

Stankowski also presents an equation for percentage imperviousness in terms of population density. His observed range of I was from 1 to 72%, but only 6 of his catchments had values greater than 35%. Using I = 1 and 35 to represent rural and fully urbanised conditions, equations (3.8) predict increases of 140% and 65% in the 2 and 100 year flood peaks.

* Dempster (1974) developed equations for 2, 5, 10, 25, 50 and 100 year floods for the Dallas region of Texas. His analysis was basically a rainfall-runoff modelling exercise (see section 3.3.5) but he presents no information on variation of model parameters with urbanisation, rather variation of derived T-year flood with urbanisation. He fitted the USGS model (Dawdy *et al*, 1972) to 14 catchments, and used a 57 year rainfall record to derive synthetic flow records. Fitting the Log Pearson III distribution, he obtained estimates of the 2, 5, 10, 25, 50 and 100 year floods, which he then regressed on catchment characteristics. His equations for 2 and 100 year floods are presented below:

$$Q_2 = 369\ A^{.90}\ (L/\sqrt{S})^{-.19}\ K^{.65}$$
$$Q_{100} = 1172A^{1.02}\ (L/\sqrt{S})^{-.29}\ K^{.33} \qquad (3.9)$$

where: Q is flood peak (ft^3/s)

A is catchment area (mi^2)

L is channel length (mi)

S is channel slope (ft/mi)

and K is Carter's (1961) impermeability factor (1+.015I)

These equations predict increase in Q_2 and Q_{100} of 31% and 15% respectively as I increases from 0 to 35%. These results are much smaller than Johnson and Sayre's, though it should be borne in mind both studies are based on synthetic data. Dempster explains the difference between Houston and Dallas as due to the steeper slopes, shallower soil, and greater channel conveyances in the Dallas region before urbanisation. Following Dempster's study, the USGS model has been used to estimate T-year floods in rural and urban catchments by several workers (Wibben, 1976; Thomas and Corley, 1977; Golden, 1977). Lichty and Liscum (1978) use the results from several such studies to relate T-year floods to lag time, infiltration factor and imperviousness. However no method of estimating these parameters for ungauged catchments was given.

Hollis (1975) updated and adjusted Leopold's (1968) summary paper. He presented a graph (Fig. 3.4) of the ratio of the pre-urban to post-urban discharge for various percentages of paved area and various return periods. His graph suggests 35% impervious area would increase 2 year flood by about 275% and 100 year flood by about 80%.

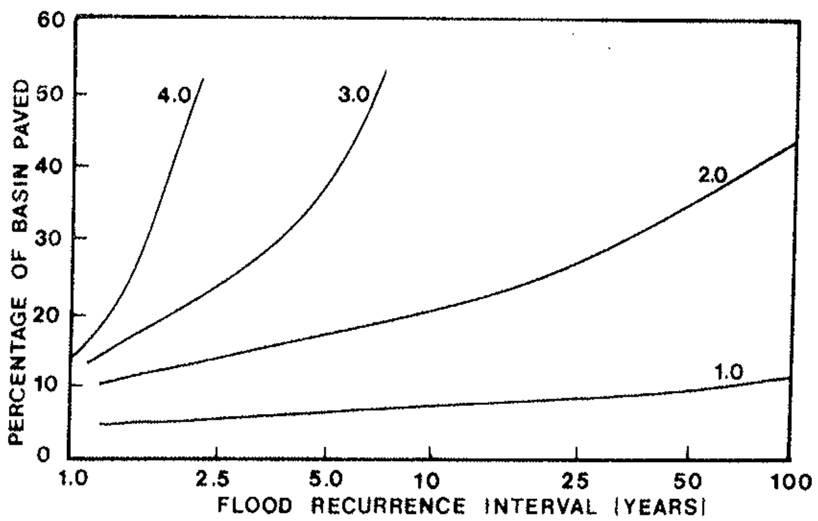


FIGURE 3.4 Effect of urbanisation on floods of different recurrence intervals
(from Hollis, 1975)

Doehring et al. (1975) used data from 5 catchments in New England in a before and after, split record analysis. They fitted the Log Pearson III distribution to each half record and determined the increase in 100 year flood as a proportion of the original (not necessarily rural) mean annual flood. They then presented a linear (additive) regression equation relating the increase to morphometric parameters and a land-use-change parameter defined as the change in proportion urbanised multiplied by the original proportion rural. (An additive equation with this land use parameter predicts greater effects of urbanisation when the original condition is completely rural - a feature common to the more usual multiplicative equation for T-year flood on percentage imperviousness). Smith and Doehring (1978) extended the analysis to include 18 catchments, deriving equations for both 100 and 50 year floods. Their equations predict large increases of 650% and 850% for 50 and 100 year floods as urban area increases from 0 to 100%. These large increases may be due to applying their equations beyond the somewhat restricted range of land-use-change used in analysis. However, the unusual result they obtained of an increasing effect of urbanisation on rarer floods would be predicted over the full range of land use change.

Weiss (1975) used data from 94 rural catchments in Connecticut to relate 2 and 100 year floods to catchment area, basin ratio ($L/\sqrt{S}$) and rainfall depth of duration 12 or 24 hours for the same return period. Using data from a further 11 urbanised catchments, he related lag time to basin ratio. He found lag times about five times as long as Carter (1961) though some confusion exists in his paper over conversion factors to metric units. He found that his rural catchment equations adequately predicted urban 2 and 100 year floods, providing $L/\sqrt{S}$ was increased to the equivalent value for a completely rural catchment having the same lag time.

Cech and Assaf (1976) analysed data from the Houston area to develop a contour map of mean annual flood per unit area. Inclusion of data from urban catchments yielded a 3 to 5 times increase in contour values in some areas.

Wibben (1976) used both observed and synthetic data from 14 catchments in Tennessee to develop equations for 2,5,10,25,50 and 100 year floods. Like Dempster (1974) he used the USGS model to generate 72 years of synthetic data and fitted the Log Pearson III distribution to obtain estimates of T-year floods. These estimates he combined with estimates obtained directly from the observed records (averaging 11 years in length) weighting according to the expected relative errors. He then derived equations for T-year flood and lag time, but found no significant relationship of either with the extent of impervious area.

Williams (1976) reported on the Wairau Creek watershed in New Zealand. An increase in urban proportion from about 30% to 65% had increased peak stages by about 60% and increased the incidence of bankfull discharge from about three times a year to about five times a year. Typical hydrograph base times were approximately halved.

Gundlach (1978) described the use of the US Corps of Engineers (1973) model to adjust observed flood peaks for urbanisation. Using standard storm profiles he developed a simple look-up graph for adjusting the flood peak in any year to current day conditions.

* The above summary of flood-frequency technique applied to urbanising catchments has shown a wide variation in the effect of urbanisation on mean annual and 100 year floods. Perhaps average values for the increases would be about 275% and 100% respectively. In general, more work has been done on the effect on mean annual flood than on rarer floods. Indeed, most of the work on rarer floods is based on data which is to some extent synthetic. The main problem with accurate estimation of the effect of urbanisation on rare floods is the shortage of long stationary records. Some workers have tried to remove the effects of

urbanisation from records subject to continuing urban development. However, while these attempts are laudable in their use of local data, from a statistical standpoint their procedures are indefensible. As discussed earlier, since flood-frequency information for urban catchments is generally scarce, rainfall-runoff modelling has been used extensively in its place. These techniques are discussed in the following section.

3.3 Rainfall-runoff modelling techniques

Rainfall-runoff models have been developed to permit estimation of design floods when suitable flow records are not available, making use of the more generally available and more easily transferable rainfall records. Methods range from simple peak-flow formulae to complex models based on continuous simulation of the water balance and hydraulic routing of surface water runoff. In general, rainfall-runoff models are well suited to studies of land use change since the rainfall process may be considered unaffected. Changes in flood magnitudes may therefore be related to changes in catchment response which may in turn be related to changes in the catchment's physical characteristics. However, to estimate the flood of some required return period, the method relies (in general) on the choice of a suitable "design storm" and percentage runoff. An unsuitable choice could lead to large errors of estimation.

In modelling the transformation of rainfall to runoff there are two basic problems to solve; how much water runs off? and how quickly? These two problems are normally considered separately. Firstly, total rainfall is separated into "effective" rainfall (ie that part which yields quick response - surface or shallow subsurface runoff) and "losses" (ie that part which is lost to interception, infiltration, and groundwater recharge). Secondly, the effective rainfall is "routed" to the basin outlet using a "direct" (quick response) runoff model. Finally, a generally small level of "baseflow" (slow response) runoff is added, representing the groundwater response (ie that response related primarily to the catchment condition before the rainfall event, but also incorporating the effect of any groundwater recharge during the event). Three separate sub-models are required, therefore: a losses model to estimate the volume and distribution of rainfall losses through the storm: a direct runoff model to route effective rainfall to direct runoff and a baseflow model for groundwater response. Each of these processes (losses, direct runoff, baseflow) may be affected by urbanisation, and methods for predicting the effects are considered in Sections 3.3.1 to 3.3.5. However, in analysing observed data, it is impossible rigorously to separate losses from effective rainfall or baseflow from direct runoff. Consequently an arbitrary choice of separation model is made; some commonly used procedures are outlined in figure 3.5 (note: the baseflow separation defines the volume of direct runoff, which in turn defines the volume of effective rainfall). Having obtained the separated data, a model is sought to represent the conversion of effective rainfall to direct runoff. The form of this model must depend to some extent on the arbitrary separation procedures adopted, a direct runoff model developed using one particular combination of separation models will not be truly comparable with another model developed using another combination. This aspect has not,

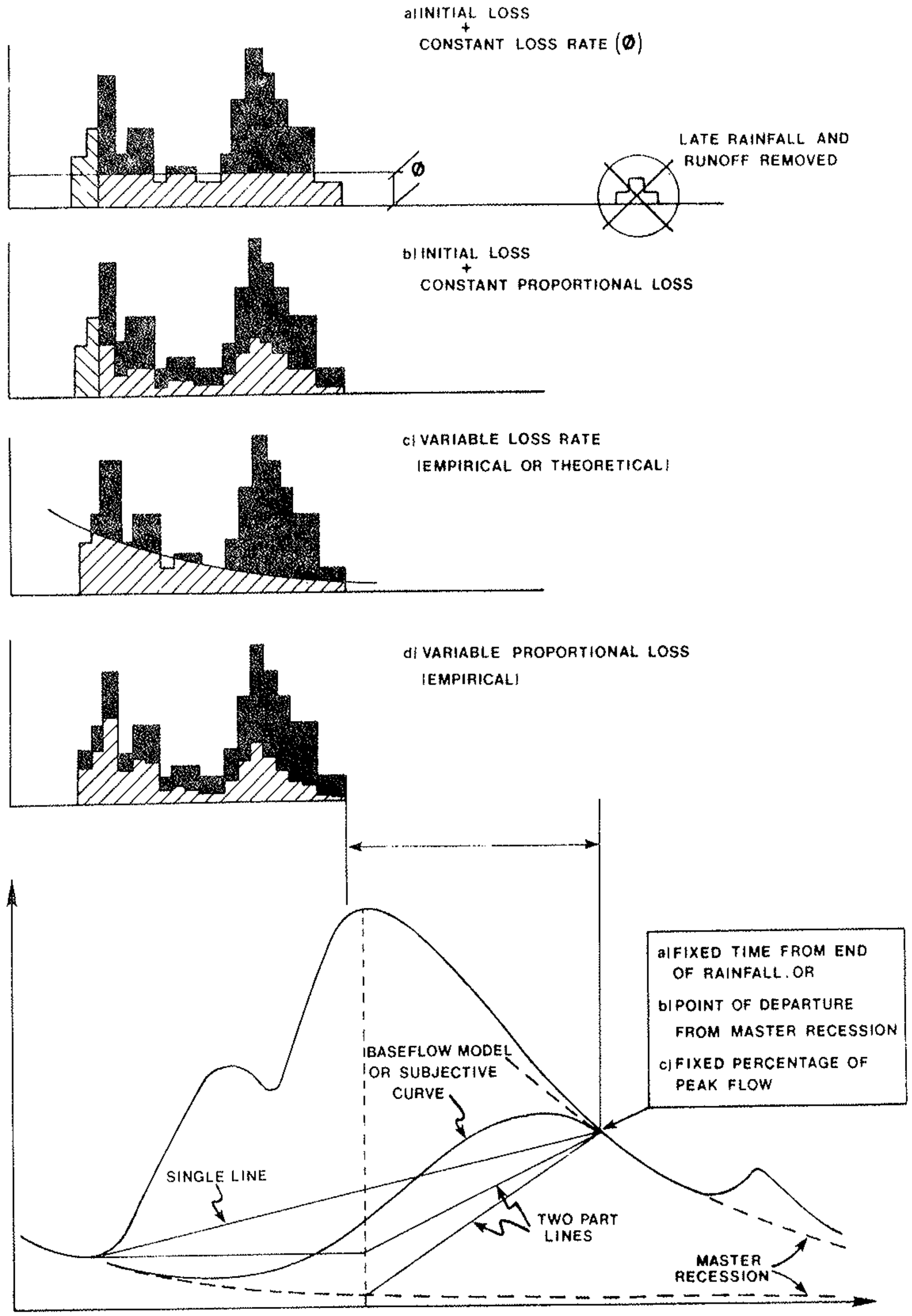


FIGURE 3.5 Some commonly used separation techniques, showing range of shapes obtained for net rainfall and surface runoff

however, been considered until relatively recently when, with the development of computer based models, all three submodels may be fitted simultaneously. In this report, loss models, baseflow models and direct runoff models will be considered separately, though mention will be made where a particular combination of models is recommended. In general, far more work has been done on the effects of urbanisation on direct runoff than on losses or baseflow. For convenience of presentation, however, losses and baseflow separation are considered first followed by direct runoff models in three classifications - the Rational method, unit hydrograph methods, and catchment simulation methods.

3.3.1 Effective Rainfall Separation

A. General Introduction

Effective rainfall separation involves estimation of the total volume of rainfall losses and also the distribution of the losses in time. In analysis of observed data, the total volume of loss is determined as the volume of rainfall minus the volume of direct runoff, and it is only necessary to distribute that loss in time. In design it is necessary to estimate both the volume of loss and its distribution in time.

Total volume of loss is traditionally considered using the runoff coefficient, C, or percentage runoff, PR , where

$$C = Q/P = (P - L)/P = PR/100 \qquad (3.10)$$

Q is runoff volume expressed as a depth over the catchment

P is rainfall depth

L is total loss

Values obtained for C depend on a number of factors including soil type, cover type (eg forest, pasture, crops, urban) and antecedent conditions. Typical values may however be found in standard hydrological texts (eg Chow, 1965; Gray, 1970) from which tables 3.1 and 3.2 have been abstracted

TABLE 3.1 Runoff coefficients for rural catchments

Soil type	Cover Type		
	Cultivated	Pasture	Woodland
Sands & Gravels - Above average infiltration	0.20	0.15	0.10
Loams, etc - average infiltration	0.40	0.35	0.30
Clays, or shallow rocky soils - below average infiltration	0.50	0.45	0.40

TABLE 3.2 **Runoff coefficients for urban catchments**

Description of area	Runoff Coefficient
Central business area	0.70 to 0.95
Local business area	0.50 to 0.70
Light industrial area	0.50 to 0.80
Heavy industrial area	0.60 to 0.90
Residential partments	0.50 to 0.70
Residential detached units	0.40 to 0.60
Residential suburban	0.25 to 0.40
Parks, cemeteries, open space, etc	0.10 to 0.30

The runoff coefficient gives total loss, but is also frequently used for distributing the loss in time, assuming a constant proportion of rainfall in any time period yields runoff, and a constant proportion is lost. The assumption of constant proportional loss is particularly common for urban catchments, where the further assumption of zero loss from paved areas and 100% loss from pervious areas is often made. Other frequently used methods of distributing the loss include constant loss rates, initial loss plus constant or variable loss rates, and variable proportional loss. Fig 3.5 shows some of these effective rainfall separation techniques. One particular technique developed for UK conditions (NERC, 1975) involved estimating overall percentage runoff in terms of a soil index, the total storm rainfall, and a "Catchment Wetness Index (CWI)". Losses were then distributed during the storm using a variable proportional loss dependent on the value of CWI which was updated hourly during the storm.

The separation techniques considered above treat loss and its distribution in time independently. However, several procedures based on loss rate curves have been proposed to treat volume and distribution of loss simultaneously. Horton (1933, 1940) considered the processes that together made up the losses and he proposed an initial loss to represent interception and depression storage and then a negative exponential equation to represent infiltration:

$$f_t = f_c + (f_o - f_c)\, e^{-kt} \qquad (3.11)$$

where f_t is the infiltration rate at time t

f_o, f_c are the initial and long term infiltration rates

and k is a decay constant.

Values of f_c and k depend on soil properties and may be defined from sprinkler plot data; f_o depends on antecedent wetness. Equation (3.11) or similar empirically defined loss rate curves have been used by a number of workers (eg Horner and Jens, 1942; Hicks, 1944; Holton and Kirkpatrick, 1950; Tholin and Keifer, 1960), where infiltration is only considered over the pervious area of the catchment. Besides equation (3.11) two other infiltration formulae have frequently been used: Philip's (1957) equation:

$$f_t = St^{-\frac{1}{2}} + A \qquad (3.12)$$

where S and A are constants

and Holton's (1961) equation:

$$f_t = A\,(S - F)^n + f_c \qquad (3.13)$$

where S is the soil moisture capacity
F is cumulative infiltration
A and n are constants
and f_c is the long term infiltration rate.

Such loss curve techniques, however, require estimation of several constants including the initial interception loss. Moreover, conceptual problems in their use include: scaling up data from sprinkler plots to represent whole catchments; choosing an initial starting point on the curve depending on antecedent conditions; and considering periods when rainfall intensity is less than infiltration rate. To overcome some of these problems the Soil Conservation Service (SCS) of USDA (1957) have developed a simpler technique for estimating both interception and infiltration loss (or more exactly for estimating runoff volume):

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (3.14)$$

where Q is total runoff
P is total rainfall
and S is potential infiltration, given in terms of runoff curve number, CN as

$$S = (1000/CN) - 10 \qquad (3.15)$$

Values for CN are presented (USDA, 1957; Chow, 1965) for a range of soil and cover types assuming standard antecedent conditions. The CN value relevant to the catchment can thus be determined as a weighted average over the soil and cover types present (hence the name "soil-cover-complex" method). The standard condition CN value may then be adjusted for antecedent conditions into one of three categories using relationships also presented.

B. The Effects of Urbanisation

Estimation of the effects of urbanisation on effective rainfall has tended to be left to intuition. Snyder (1958) expressed percentage runoff as the linear combination of the rural percentage runoff from the unpaved areas (which he estimated from the monthly water balance) and 100% runoff from the paved areas. Tholin and Keifer (1960) after subtracting separate initial losses from the rainfall profile for paved and unpaved area, assumed zero infiltration loss from paved areas, and used Horton's equation (3.11) for loss rate from unpaved areas. Carter (1961) produced an equation similar to Snyder's (1958) equation, but assumed 30% runoff from pervious surfaces and 75% runoff from impervious surfaces. Several workers, including Wiitala (1961), Dagg and Pratt (1962), Sawyer (1963), Harris & Rantz (1964), Antoine (1964), Espey et al (1965), Lull and Sopper (1969), Seaburn (1969), Feddes *et al* (1970), Wallace (1971), Albrecht (1974) and Taylor (1977) have all reported increases in runoff with urbanisation, but have not generalised their results for use in ungauged catchments.

Brater (1968) used a plot of runoff against rainfall to identify initial losses and the "Hydrologically Significant Impervious Area" - given as the minimum ratio of runoff to rainfall-less-initial-losses. Average infiltration from the pervious area could then be found. Several other workers have also considered Hydrologically Significant Impervious Area, typically finding it equivalent to ½ to ¾ of the total impervious area. The difference is usually attributed to the remainder of the area not being directly connected to the sewer system.

Narayana *et al* (1968) presented equations for the parameters of Horton's (1940) loss curve based on impervious fraction and a characteristic impervious length factor. These equations, however, are based on one catchment only and are not readily transferable to others.

Stall *et al.* (1970) interpolated between the SCS soil-cover-complex method for rural conditions and 100% runoff from the directly connected impervious area for fully urbanised conditions. Miller and Viessman (1972) used loss rates based on percentage imperviousness for rainfalls less than 1.5 inches, but for rainfalls greater than 1.5 inches they used the SCS soil-cover-complex method to allow for runoff from pervious surfaces. Kao *et al.* (1973) also used the SCS method in analysing runoff from three urban catchments in Arizona, and concluded the method was sensitive enough for
* determining the effects of urbanisation. USDA (1975) themselves present soil-cover-complex curve numbers for urban catchment applications. (see Table 3.3).

Hossain et al (1974) compared the performances of five rainfall excess models on an urban catchment in Indiana. Two models (the Multicapacity Basin Accounting Model and the Antecedent Retention Index) gave volume of rainfall excess alone, and the other three (the MIT model, the Minnesota University Model, and Holtan's equation) gave infiltration rate distributions. They concluded the Antecedent Retention Index predicted volumes best, while the MIT model was best for loss rate distribution.

* NERC (1975) analysed percentage runoff data from 132 catchments in the UK, 24 of which had urban fractions greater than 10%. They present the following equation:

TABLE 3.3 Curve numbers for different land uses and soil groups

Land Use Description	SOIL GROUP (see below) A	B	C	D
Cultivated Land:	72	81	88	91
Pasture:	39	61	79	80
Woodland and Forest:	25	55	70	77
Open Spaces, Parks, etc: 75% or more grass	39	61	74	80
50% to 75% grass	49	69	79	84
Residential: 8 houses/acre (65% impervious)	77	85	90	92
4 houses/acre (38% impervious)	61	75	83	87
2 houses/acre (25% impervious)	54	70	80	85
Industrial Districts: (72% Impervious)	81	88	91	93
Commercial/Business Area: (85% impervious)	89	92	94	95
Paved Roads, Car Parks, Roofs, Driveways etc.	98	98	98	98

SOIL GROUP
A - Low runoff potential, deep well drained sands and gravels
B - Moderately good depth and drainage, moderately coarse texture
C - Moderately poor depth and drainage, moderately fine texture
D - High runoff potential, shallow poorly drained clays.

$$PR = 95.5\ SOIL + .22\ (CWI-125) + .10\ (P-10) + 12.0\ URBAN \quad (3.16)$$

where SOIL is an index of soil type varying between 0.1 and 0.5
CWI is an index of catchment antecedent wetness
P is total rainfall depth
and URBAN is the fraction of the catchment urbanised

From this summary of research and methods for effective rainfall separation, it can be seen that relatively little general work has been done, but that the problem is often left to intuition. It is common to assume 100% runoff from directly connected impervious area (or 50 to 75% runoff from total impervious area) and the original rural percentage runoff from pervious surfaces. While simple procedures such as runoff coefficients, ϕ - indices or runoff curve numbers are frequently used with simple direct runoff models, the more complicated catchment simulation models will often use Horton's, Phillips, or Holton's equation, coupled with a continuous running water balance computation for soil moisture condition. In this case, individual fitting to observed catchment data to obtain the model parameters is almost always necessary.

3.3.2 Baseflow Separation

Baseflow usually contributes only a small component to the peak of the flood hydrograph, though its volume contribution may be more significant. In rainfall-runoff analysis, baseflow is separated in order to find the direct runoff hydrograph and hence the volume of effective rainfall. Several baseflow separation procedures were shown on figure 3.5. In design, however, the baseflow contribution is often ignored or handled only very crudely (eg - as a constant discharge). The volume contribution of baseflow has often been considered in water balance studies, but rarely has this information been used in flood studies. More recently, however, the computer-based catchment simulation models often use a running water balance to evaluate infiltration, and then route infiltration to groundwater recharge and hence to baseflow.

A number of workers have considered the effect of urbanisation on the baseflow contribution to the annual water balance (Sawyer, 1963; Harris and Rantz, 1964; Franke and McClymonds, 1972, ASCE/UNESCO, 1974; Kuprianov, 1977) and simulation models have been used to estimate changes in baseflow response (Hollis, 1970). However, the effect of urbanisation on the baseflow contribution to peak discharges has not generally been considered.

3.3.3 Direct Runoff - The Rational Method

Sections 3.3.1 and 3.3.2 have considered losses and baseflow models for urbanising catchments. Sections 3.3.3, 3.3.4 and 3.3.5 will consider direct runoff routing models and complete simulation models. The Rational Method is one of the simplest direct runoff models. Variously attributed to Mulvaney's (1850) work on natural catchments, and Kuichling's (1889) and Lloyd Davies' (1906) work on urban catchments, the method relates peak discharge per unit area, Q/A, to average rainfall intensity I in some critical duration, ie.

$$Q = CIA \qquad (3.17)$$

where C is the Rational Coefficient.

Assuming uniform rainfall and constant percentage runoff, peak discharge will occur when sufficient time has elapsed that runoff from the most remote part of the catchment reaches the outlet, and the whole catchment is contributing to the flow. This time, the time of concentration (T_c), is usually taken as the critical duration in determining I. Under these conditions the Rational Coefficient C is equivalent to the volume of runoff coefficient (considered in Section 3.3.1) times a conversion factor dependent on the units used. In imperial units, with Q(ft^3/s), I (in/hr) and A (acres) the conversion factor should be 1.008, but this is usually rounded down to 1.0. For A in square miles the factor should be 640 x 1.008 = 645. In metric units with Q(m^3/s), I(mm/hr), and A (km^2) the factor is 0.278. The method makes no explicit allowance for either losses or baseflow.

Applied to individual storms, a large scatter in observed C-value is obtained since C has to account for (i) variation in percentage runoff between storms, and (ii) variation from uniform rainfall intensity coupled with variation in flow times from different parts of the catchment. To improve consistency, several workers have separated the Rational Coefficient into a percentage runoff factor and a routing factor (Snyder, 1958; Chow, 1962; Aitken, 1968; Da Costa, 1970; Kadoya, 1973; Institute of Hydrology, 1979). However, it is more usual nowadays to consider the Rational Method, not as a rainfall-runoff model for individual storms, but as a statistical model relating T-year flood peak to T-year rainfall.

For large rural catchments, the variation of C and T_c has made estimation difficult and use of the Rational Method has therefore been largely superseded. In small or urban catchments, however, C and T_c are generally less variable since the routing effect is less significant and more adequately accounted for by T_c. Thus C may be put equal to the runoff coefficient (see Section 3.3.1 Tables 3.1 and 3.2) and values for T_c may be estimated from sewer data or empirical formulae like that due to the Kirpich (1940):

$$T_c = .00013\ ({}^{L}/\sqrt{S})^{0.77} \qquad (3.18)$$

where T_c is time of concentration (hr)
L is overland flow path (ft)
and S is slope (ratio)

Similar values for the Rational Coefficient and formulae for T_c may be found in standard hydrological texts (eg Chow, 1965; Gray, 1970) and in specialist review papers (eg McPherson, 1969; Colyer and Pethick, 1976). However, the Rational Method yields only a peak flow estimate, and little research has been done on applying the model to estimating the effects of urbanisation.

3.3.4 Direct runoff - unit hydrograph and area-time methods

A. General Introduction

To improve consistency of flood peak estimation, and to yield estimates of the complete hydrograph, unit hydrograph and area-time methods have been developed by hydrologists and drainage engineers respectively. The two methods are essentially the same, but the unit hydrograph method is based on analysis of observed rainfall and runoff data, while area-time methods are based on a physical concept of how a catchment behaves. A full discussion of the theory and assumptions of unit hydrograph theory may be found in standard hydrological texts (eg Chow, 1965; Gray, 1970; Wilson 1974), but a brief summary of the concepts and analytical procedures is given below.

The unit hydrograph method developed by Sherman (1932) considers the direct runoff hydrograph due to a unit depth of effective rainfall in unit duration. If this unit hydrograph can be found, the direct runoff hydrograph due to any complex storm may be obtained using the "linear" principles of proportionality and super-position (see fig. 3.6), which may be expressed as the "convolution" equation:

$$q_t = \sum_{\tau=1}^{t} p_\tau . u_{t+1-\tau} \tag{3.19}$$

where q_t is the direct runoff ordinate at timestep t

p_τ is the effective rainfall depth in the timestep $\tau-1$ to τ

u_τ is the unit hydrograph ordinate at timestep τ

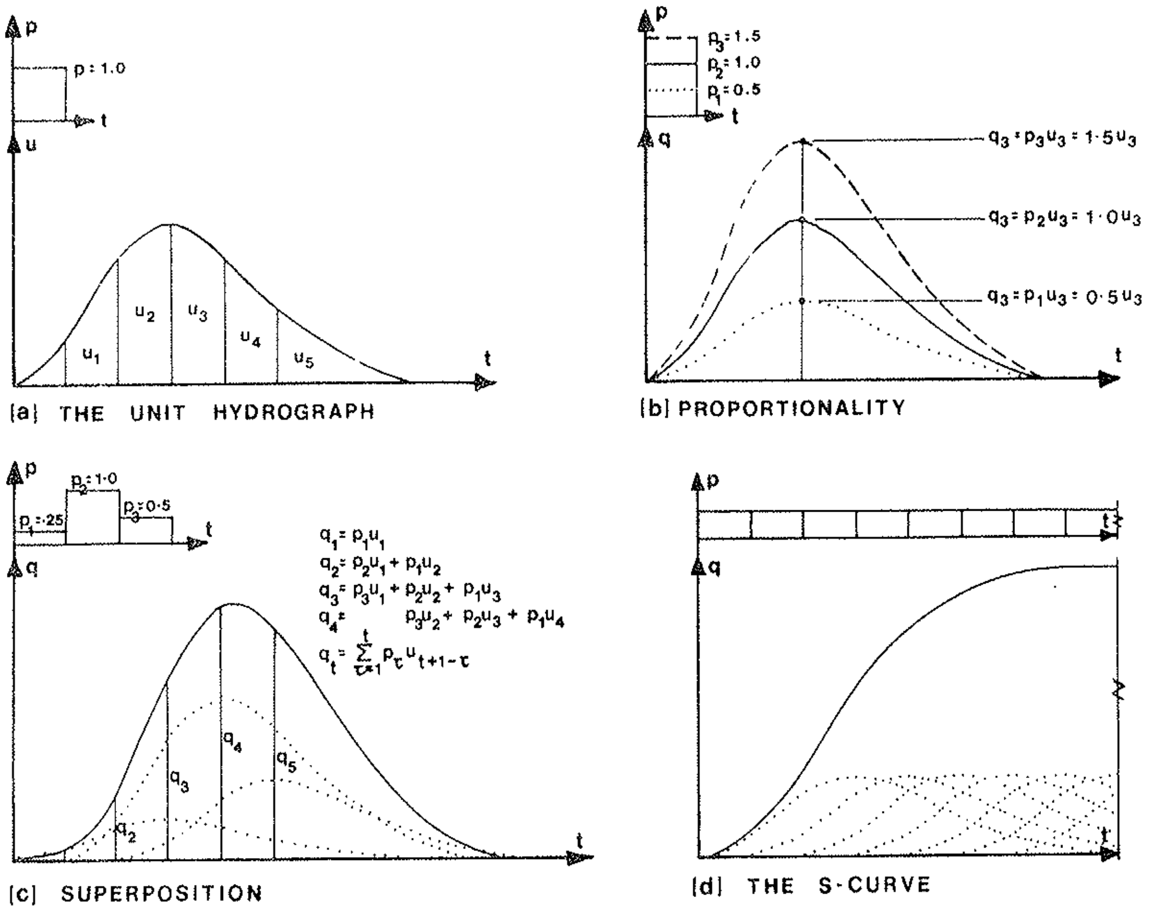


FIGURE 3.6 The unit hydrograph concept

To derive a unit hydrograph first direct runoff must be separated from total runoff and effective rainfall separated from total rainfall, and then equation (3.19) must be solved for u_τ. However, expanding equation (3.19) gives in general an inconsistent set of simultaneous equations in u_τ and some error minimising technique is necessary for solution. Such error minimising techniques may be divided into two types: (i) analytical "black box" techniques such as matrix inversion on harmonic analysis, where no assumption is made *a Priori* about the overall shape of the unit hydrograph (see March and Eagleson, 1965; Laurenson and O'Donnell, 1969; Delleur and Rao, 1971); and (ii) conceptual model techniques where the parametric form of the unit hydrograph is specified beforehand (eg a triangle) and the parameters (eg peak, time to peak) chosen to give the best fit to the data (see Clark, 1945; Nash, 1957; Dooge, 1959; Kulandaiswamy, 1964).

The unit hydrograph derived corresponds to a unit storm duration given by the timestep used in equation (3.19). To derive the unit hydrograph due to some other duration the S-curve may be used (see fig. 3.6). The S-curve describes the catchment response from zero flow to steady state under constant intensity effective rainfall and is obtained by superimposing successive unit hydrographs. Conversely, therefore, the unit hydrograph of any duration D may be found by subtracting two S-curves a distance D apart and scaling the resultant hydrograph to unit volume. Taken to the limit (D=O) this procedure amounts to differentiation, and thereby defines the Instantaneous Unit Hydrograph (IUH) - the hydrograph due to unit depth of rain falling instantaneously over the catchment. The shape of the IUH is therefore given as the slope of the S-curve.

The S-curve is also of interest because, by describing the zero flow to steady state response, it is directly analagous to the area-time curve developed primarily for sewer design (see Colyer and Pethick, 1976). The area-time curve is the distribution (in time) of area within the catchment having flow times to the outlet less than or equal to successive times, t. The area-time curve therefore represents pure translation (concentration) under "steady state" conditions, while the S-curve is derived from observed "unsteady" flows and incorporates both translation and attenuation. In the same way that the S-curve may be used to define an IUH, so the "cumulative" area-time curve discussed above may be used to define an "instantaneous" area-time curve. This curve also represents only translation, but has been used in place of a unit hydrograph (Clark, 1945; Watkins, 1962), with a subsequent storage routing to represent attenuation.

Unit hydrograph theory has been applied extensively to rural catchments. By separating consideration of losses and baseflow the characteristics of the routing process may be separately identified and related to catchment characteristics (Snyder, 1938; USDA, 1957; Nash, 1960; US Army Corps of Engineers, 1963; NERC, 1975). Unit hydrographs may also be used to define the routing component of the Rational Coefficient (Snyder, 1958; Chow, 1962; Aitken, 1968; Da Costa, 1970; Institute of Hydrology, 1979). Unit hydrograph theory is not however, in general used for sewer design, though it has been used to analyse sewered catchment data (Horner and Flynt, 1936; Snyder, 1958; Eagleson, 1962; Watkins, 1962). Sewer design requires flow estimates at successive points down the sewer network, which would require derivation of successive unit hydrographs. In these circumstances the subcatchment approach is more applicable, routing upstream inflows down to the next design point and adding in a new surface runoff hydrograph for the intermediate areas. Unit hydrographs have been used to estimate the intermediate area hydrographs, but such areas are generally very small (less than 1 ha) and of very rapid response. The unit hydrographs are not therefore easily compared with those from complete catchments.

B. The effects of urbanisation

Although unit hydrograph theory has rarely been used for sewer design, several workers have applied it to estimating the downstream effects of urbanisation. Unfortunately, there are almost as many techniques as there are workers, and comparison between studies can be difficult. This is particularly true of the definition adopted for lag time - used as a scaling parameter for the unit hydrograph. Different workers have defined lag time from (i) total rainfall and total runoff, (ii) effective rainfall and direct runoff, or (iii) the unit storm and the unit hydrograph. Furthermore lag time may refer to time to peak, t_p, time to centroid, t_c, or time to median (50% of volume), t_m. Also time may be measured from start of rainfall or centroid of rainfall (in this review the superscript ' indicates time measured from the centroid). Great care should therefore be taken in interpreting the results of separate studies. A summary of simple lag time relationships is given in Appendix 3.

* Many workers have used Snyder's (1938) synthetic unit hydrograph equations relating peak and time to peak to catchment characteristics.

$$t'_p = C_t (LL_{ca})^{0.3} \qquad (3.20)$$

$$t_r = t_p/5.5 \qquad (3.21)$$

$$q_p = C_p \; 640/t'_p \qquad (3.22)$$

where t'_p is the lag time from centroid of unit rainfall to peak of the unit hydrograph (hr)

L is main channel length (mi)

L_{ca} is distance along main channel to point nearest centroid of catchment (mi)

t_r is unit storm duration (hr) - (unit storm depth - 1 in)

q_p is peak of unit hydrograph ($ft^3/s/mi^2$)

C_p & C_t are the Snyder coefficients

The factor 640 in equation (3.22) converts q in $ft^3/s/acre$ to $ft^3/s/mi^2$. For q_p in $m^3/s/km^2$ and a unit storm depth of 1 cm the factor would be 2.75. Values of C_p and C_t for rural catchments show large regional variation ($.15 \lesssim C_p \lesssim .95$; $.4 \lesssim C_t \lesssim 8.0$), but hydrograph data allowing local estimation of the coefficients is given for many regions of the USA by the US Army Corps of Engineers (1959, 1963); equations for estimating unit hydrograph width at 50% and 75% peak discharge are also presented. The coefficient C_p gives the peak of the unit hydrograph, but also, because the unit hydrograph has unit volume, gives information on shape. Writing equation (3.22) as

$$\frac{q_p t'_p}{2} = 320\ C_p \qquad (3.23)$$

it can be seen that C_p is closely related to the volume under a straight line approximating to the rising limb of the unit hydrograph (note t'_p is time from centroid of unit storm, not start of rise). Considering flow in ft^3/s and time in hours, the total volume under the unit hydrograph is 645, and thus the ratio of the volume under the rising limb to the total volume is given approximately at $C_p/2$. A C_p value near 1 therefore implies a fairly symmetrical unit hydrograph while a value near 0.25 implies about 7 times as much volume under the falling limb as under the rising limb.

Several workers have presented C_p and C_t values for urban catchments. Van Sickle (1962) analysed data from Brays Bayou in Houston, Texas, showing that for rural conditions C_p varied from 0.33 to 0.19 (depending on weed growth in the channel) and C_t was approximately 2.7. However when approximately half urbanised C_p reduced to .11 and C_t to 0.3. Van Sickle also noted from equations (3.20) and (3.22) that q_p was proportional to the ratio C_p/C_t. This had changed from 0.12 to 0.37 due to the urban development. Thus urbanisation had decreased t'_p by approximately 90% and increased q_p by approximately 200%.

* Eagleson (1962) analysed data from 5 urban catchments in Louisville, Kentucky, ranging in imperviousness from 30 to 80%. He obtained C_p values ranging from .24 to .63 and C_t values ranging from .21 to .32. Some trend towards progressive change with percentage imperviousness was present. Eagleson also considered Linsley et al's (1958) modified form of equation (3.20).

$$t_m = C_t \left(\frac{LL_{ca}}{\sqrt{S}}\right)^{.38} \qquad (3.24)$$

where t_m is lag time from start of rain to passage of 50% of runoff. Eagleson found for his sewered catchments an average value for C_t of 0.18 compared with Linsley et al's values of 1.2, 0.72, and 0.35 for mountain, foothill and valley floor drainage area respectively. Finally, replacing t_m in equation (3.24) by Snyder's lag time t'_p he obtained a revised C_t value for his urban catchments of 0.067. He concluded urbanisation had decreased t'_p by up to 70%.

* Van Sickle (1969) reported an extension of Eagleson's (1962) analysis including data from Texas. Mindful of Carter's (1961) lag time graph (see section 3.2) he evaluated separate C_t values for use in equation (3.24)

for 6 differing degrees of development and sewerage, ranging from $C_t = 0.9$ for woodland or pasture to $C_t = 0.065$ for extensive storm sewerage and channel improvement (see Figure 3.7). His results predict a reduction in lag time of 93%. He went on to present a revised technique redefining the basin characteristic $(LL_{ca}/\sqrt{S})$ in terms of an areal average length and slope. His procedures have been used by Curtis *et al* (1964) and Hare (1970).

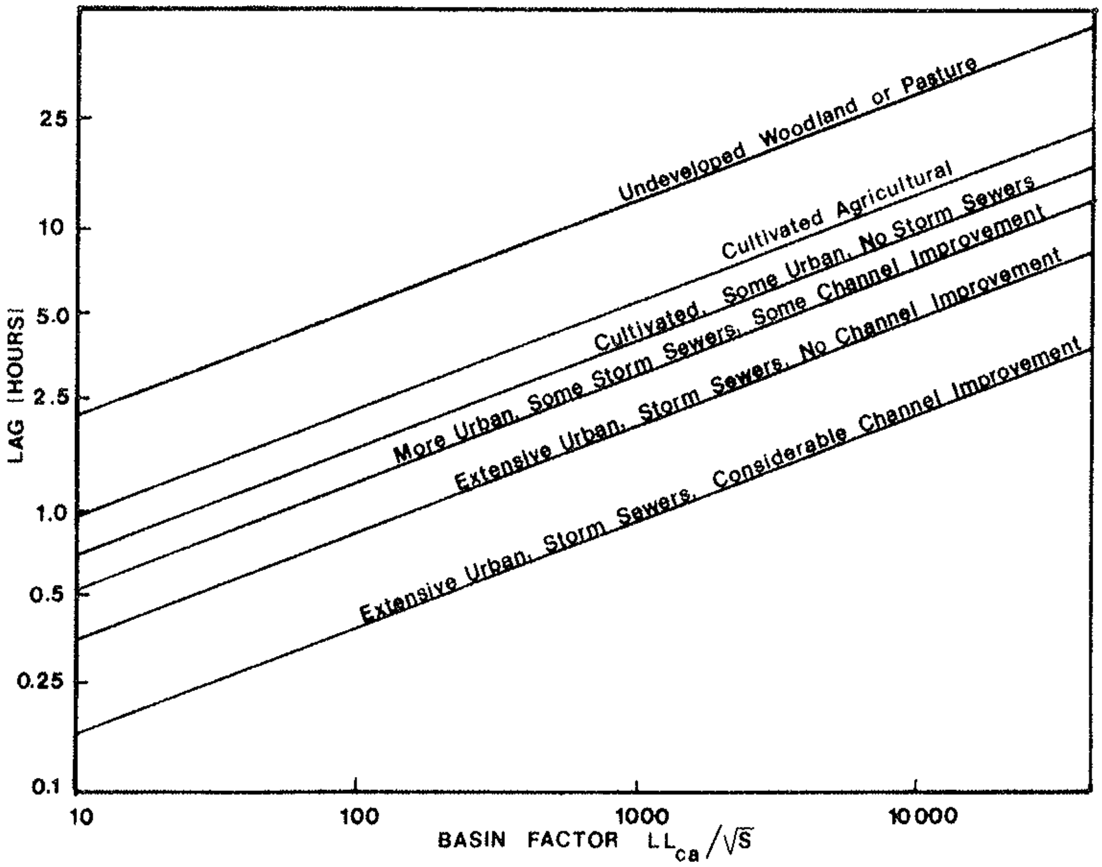


FIGURE 3.7 **Lag time against basin factor for various degrees of urbanisation**
(from Van Sickle, 1969)

Crippen (1965, 1969) analysed the effect of development on Sharon Creek, California. He found Snyder's C_p had increased from .26 to .38, but that due to the upstream location of the urbanisation t'_p had not changed. He did however consider other measures of lag, and he showed the centroid to centroid lag time t'_c had reduced by 22%. Presenting his results in the same form as Carter (1961) - see Section 3.2 - he obtained

$$t'_c = C_t \, (L/\sqrt{S})^{.6} \qquad (3.25)$$

with C_t values of 4.7 and 3.7 for pre- and post-development conditions (nb. Carter's work gave values of 3.1 and 1.2 for natural and partially developed conditions.) Crippen in fact presented values of 3.7 and 2.6, but these were for the centroid to median lag time t'_m, not centroid to centroid t_c'.

Weight McLaughlin Engineers (1969) have presented values for Snyder's C_p and C_t based on data from Denver, Colorado "and elsewhere". They recommended C_p values of .55 and .45 and C_t values of .35 and .25 for percentage imperviousness increasing from 20 to 60. These coefficient values predict relatively small effects of urbanisation on both t'_p and q_p.

Feddes *et al* (1970) found values of Snyder's C_p and C_t for two similar basins in Texas, one rural, one 25% impervious. They found mean C_p values of about .58 in both cases, but C_t in the urban catchment was 1.03 compared with 2.55 in the rural catchment. They concluded urbanisation had decreased t'_p by about 60% and increased q_p by about 150%.

Several contributors to a US Army Corps of Engineers seminar (1970) have also presented values for C_p and C_t for various locations in the USA and for varying degrees of urbanisation.

Bleek (1975) derived C_p and C_t values for 5 catchments in Crawley, UK, 3 of which had pre- and post- urban records. He found C_p was unaffected by urbanisation at 0.51, but C_t varied with percentage imperviousness as:

$$C_t = 3.77\ I^{-.48} \qquad (3.26)$$

This equation predicts that increasing I from 1 to 30% reduces lag time by 80% and increases unit hydrograph peak by 410%.

The studies discussed above have used procedures closely related to Snyder's (1938) synthetic unit hydrograph technique. Another widely used technique is that due to Clark (1945), routing the "instantaneous" area-time curve through a "linear reservoir" having storage S directly proportional to outflow, Q.

$$S = R\ Q \qquad (3.27)$$

where R is the storage constant.

Substituting into the continuity equation over a finite time step Δt, and solving for $Q_{t+\Delta t}$ gives successive ordinates of the unit hydrograph

$$Q_{t+\Delta t} = (I_{t+\Delta t} + I_t)\ \Delta t/(2R + \Delta t) + Q_t\ (2R - \Delta t)/(2R + \Delta t) \qquad (3.28)$$

where Q_t is the ordinate at time t of the synthetic IUH and I_t the ordinate at time t of the "instantaneous" area-time curve multiplied by a unit depth of rainfall.

This synthetic unit hydrograph method has been used quite extensively, and has been included in several simulation models (see section 3.3.5) including the Stanford Watershed Model (Crawford and Linsley, 1966), the USGS model (Dawdy *et al*, 1972), and HEC -1 (US Army Corps of Engineers, 1971). In spite of this, few equations relating parameters T_c and R to catchment characteristics have been presented, primarily because in each case it is recommended the model is fitted to individual catchments using observed data. The instantaneous area-time curve has often been represented as an isosceles triange of time base T_c, but otherwise may be derived from contour maps (see Laurenson, 1964; Black and Aitken, 1977).

Snyder (1958) used Clark's synthetic unit hydrograph technique, representing the instantaneous area-time curve by an isosceles triangle of time base T_c (the time of concentration), and using a reservoir constant $R = T_c/2$. The IUH is thus defined in terms of one parameter T_c for which Snyder presented the following equation.

$$T_c = C_t (nL/\sqrt{S})^{0 \cdot 6} \qquad (3.29)$$

where n is the Manning roughness

L is channel length (mi)

S is channel slope (ft/ft)

C_t is a coefficient varying from 1.7 for natural catchments to 0.42 for fully sewered catchments

Snyder went on to use these relationships to derive improved estimates of the Rational Coefficient.

Other workers to use the method include Aitken (1968) and Sarma *et al.* (1969), but neither give values for T_c or R. Gundlach (1976), however has presented equations for T_c and T_c+R based on 15 catchments in Philadelphia.

$$T_c = 11.54 (A/S)^{\cdot 27} (1 + 0.3I)^{-\cdot 56} \qquad (3.30)$$

$$T_c + R = 17.01 (A/S)^{\cdot 24} (1 + 0.3I)^{-\cdot 61} \qquad (3.31)$$

where A is catchment area (mi^2)

S is channel slope (ft/mi)

and I is percentage imperviousness

These equations support Snyder's (1958) use of $R \approx T_c/2$, and show that increasing I from 0 to 35% reduces both T_c and R by about 75%. Wibben (1976) presented fitted values of T_c and R for 14 Catchments in

Tennessee, and he went on to relate $R+T_c/2$ to the catchment characteristic $L/\sqrt{S}$.

$$R_c + T_c/2 = 2.25\ (L\sqrt{S})^{.52} \qquad (3.32)$$

With the instantaneous area-time curve represented by an isosceles triangle, $R_c + T_c/2$ is equivalent to the centroid to centroid lag time t_c' used by Carter (1961) and others. Wibben however found no trend with urbanisation. Thomas and Corley (1977) have presented separate equations for T_c and R based on 60 catchments in Oklahoma. Only 3 catchments were urbanised, and thus no trend with urbanisation was observed. Beard (1979) has presented an equation for T_c+R based on 14 catchments in Texas

$$T_c + R = .75A^{.6} \qquad (3.33)$$

Again, percentage impervious was not a significant variable, but Beard recommended channelisation should be accounted for in deriving the area-time curve using estimates of travel time from the Manning equation.

* Espey *et al.* (1965) presented a unit hydrograph technique designed specifically to estimate the effect of urbanisation. Using data from 24 urban catchments and 11 rural catchments from all over the United States, they derived 30 minute unit hydrographs and developed equations for time to peak t_p, peak discharge q_p, time base, and unit hydrograph width at 50% and 75% peak discharge. Subsequently, Espey *et al.* (1969) built on these equations, including data from 11 urban and 6 rural watersheds from Houston, Texas. They presented the following equations in which the values of the various regression coefficients have changed quite markedly:

URBAN CATCHMENTS

$$q_p = 35000.A^{1.0}\ t_p^{-1.10} \qquad (3.34)$$

$$t_p = 16.4\ \phi\ L^{.32}\ S^{-.049}\ I^{-.49} \qquad (3.35)$$

RURAL CATCHMENTS

$$q_p = 82500.\ A^{.99}\ t_p^{-1.25} \qquad (3.36)$$

$$t_p = 2.68\ L^{.22}\ S^{-.30} \qquad (3.37)$$

where: q_p is unit hydrograph peak (ft^3/s)

A is area (mi^2)

t_p is time to peak (minutes)

L is the main channel length (ft)

S is the main channel slope (ft/ft)

I is percentage impervious cover

Φ is channel roughness factor see Table 3.4 below

TABLE 3.4 Φ classification ($\Phi = \Phi_1 + \Phi_2$)

Degree of Channel Improvement	Φ_1
Storm-sewers and Extensive Channel Improvement	0.6
Storm-sewers and Some Channel Improvement	0.8
Natural Channel Conditions	1.0
Degree of Channel Vegetation	**Φ_2**
No Channel Vegetation	0.0
Light Channel Vegetation	0.1
Moderate Channel Vegetation	0.2
Heavy Channel Vegetation	0.3

These equations have been used by many workers (Seaburn, 1969; Stall *et al*, 1970; Johnson, 1970) and have proved capable of estimating pre- and post urbanisation unit hydrograph parameters in the USA and the UK. The choice of a suitable value for Φ is, however, a large area of subjectivity. Also, the rather low exponent for slope in equation 3.35 compared with equation 3.37 must lead to some doubt as to its general applicability. Considering equations 3.34 and 3.35 only, an increase in impervious area from 1% to 35% would reduce t_p by 82% and increase q_p by 580%. Hamm *et al*, (1973) followed a similar analysis to Espey *et al* using data from 37 catchments throughout the USA (including a few used by Espey). They present 3 separate sets of equations; all catchments; catchments less than 20% impervious; and catchments greater than 20% impervious. However, because of strong correlation between slope and imperviousness they found imperviousness was not generally significant.

Seaburn (1969) derived unit hydrographs for East Meadow Brook, a 31 square mile catchment on Long Island, New York. The catchment is on pervious soils and only the bottom 10 sq. mi. contributed to storm runoff. Urbanisation of this bottom third of the catchment had increased unit hydrograph peak by a factor of 2.5 inspite of the presence of several groundwater recharge basins.

Sarma *et al* (1969, 1973), and Rao *et al* (1972, 1974, 1978) have produced a series of papers comparing conceptual model and black box techniques for unit hydrographs in urban areas. The conceptual models used were the linear reservoir, the Nash (1959) cascade of equal linear reservoirs, the double linear reservoir (Holton and Overton, 1964), and the Clark (1945) model. They specified the baseflow separation technique used as a straight line between time of rise and a point on the recession 1% of the peak flow. Effective rainfall separation was by constant proportional loss after an initial abstraction of all rainfall before the beginning of the hydrograph rise. Of the conceptual models they found the linear reservoir was best for catchments less than 5 mi^2, and the Nash cascade was best for larger catchments. The linear reservoir gives an instantaneous unit hydrograph as

$$U(t) = (1/K)\ e^{-t/K} \qquad (3.38)$$

while the Nash cascade gives

$$U(t) = \left(\frac{1}{K_n \Gamma(n)}\right) \left(\frac{t}{K_n}\right)^{n-1} e^{-t/K_n} \qquad (3.39)$$

* Using data from 13 catchments in Indiana and Texas, Rao *et al* (1972) present regression equations for the optimum lag time K of the single linear reservoir, and the parameters n and K_n of the Nash cascade.

$$K = .89A^{.49}\ (1+U)^{-1.68}\ P_e^{-.24}\ D^{.29} \qquad (3.40)$$

$$n = 1.43A^{.07}\ (1+U)^{-1.04}\ P_e^{-.16}\ D^{.15} \qquad (3.41)$$

$$K_n = .58A^{.39}\ (1+U)^{-.62}\ P_e^{-.11}\ D^{.22} \qquad (3.42)$$

where A is catchment area (mi^2)

U is the fraction of the catchment impervious

P_e is the effective rainfall depth (in)

D is the duration of effective rainfall (hr)

These equations imply a quasi-linear approach, with model parameters constant during any one storm, but varying from storm to storm . K in equations (3.40) is directly analogous to the centroid to centroid lag time, and increasing imperviousness from 0% to 35% gives a decrease in K of 40% - a smaller effect than obtained by most workers. Rao and Delleur (1974), however, revert to "black box" techniques and present equations for peak and time to peak in terms of the same catchment characteristics.

$$q_p = 484\ A^{.72}\ (1+U)^{1.52}\ P_e^{1.11}\ D^{-.403} \qquad (3.43)$$

$$t_p = .775\ A^{.32}\ (1+U)^{-1.29}\ P_e^{-.195}\ D^{.634} \qquad (3.44)$$

Equation 3.44 predicts a reduction in time to peak of 32% for percentage imperviousness increasing from 0 to 35%. This figure is again much less than obtained by most other workers. Hossain *et al* (1974, 1978) present further equations for q_p and t_p based on only 10 of Rao and Delleur's (1974) catchments.

$$q_p = .0056\ P_e^{-.05}\ \bar{I}^{.10}\ \hat{I}^{-.16}\ L^{-1.20}\ S^{1.0}\ (1+U)^{4.75} \qquad (3.45)$$

$$t_p = 3.33\ P_e^{0.16}\ \bar{I}^{-.17}\ L^{.60}\ S^{-.21} (I+U)^{-4.31} \qquad (3.46)$$

where $\bar{I}$ and $\hat{I}$ are the mean and peak rainfall intensities (in/hr)

L is main channel length (mi)

and S is main channel slope (ft/mi)

these equations predict a reduction in t_p of 73% and an increase in q_p of 316% for a change in imperviousness from 0 to 35%. Bleek (1975) fitted the linear reservoir and Nash cascade to data from Crawley, UK and presented the following equations.

$$K = 6.53\ A^{.09}\ (1+U)^{-2.58} P_e^{\ 0.19}\ D^{0.23} \tag{3.47}$$

$$K_n = 2.58\ A^{.07}\ (1+U)^{0.68} P_e^{\ 0.12}\ D^{0.16} \tag{3.48}$$

$$n = K/K_n \tag{3.49}$$

giving a reduction in lag time of 50% for imperviousness increasing from 0 to 30%.

Wallace (1971) used the double linear reservoir to investigate the effects of urbanisation on a catchment in Georgia, USA. The catchment was already partly urbanised near the watershed divide and gave more runoff than nearby rural catchments. However, valley floor urbanisation during the study period had no significant effect on the unit hydrograph.

Several workers, particularly in Germany, have used parallel cascade models - with separate cascades for previous and impervious areas within the catchment. These, however, are considered in section 3.3.5.

* Hall (1973) analysed data from seven catchments near Crawley, UK, five of which were subject to urbanisation or channelisation effects. He separated baseflow by a straight line from start of rise to beginning of linear reservoir baseflow recession, and used a simple ϕ-index for effective rainfall. He presented a dimensionless one hour unit hydrograph (see Fig 3.8) in terms of one parameter, the centroid to centroid lag time, which he related to channel length and slope after Carter (1961) and Anderson (1970) - see Fig 3.9. His results predict a reduction in lag time of about 70% for complete urbanisation. Packman (1974) and Hall (1977) followed the same analytical procedure for two catchments in North London. One catchment was in fair agreement, but the other was subject to only infilling urban development and showed no significant effect of urbanisation. Hall subsequently modified his dimensionless unit hydrograph shape and the technique is included in a design guide for flood storage ponds, published in the UK by CIRIA (1980).

Hollis (1974) derived unit hydrographs for Canons Brook, Harlow UK, for pre and post urbanisation conditions. He found 80% urban development had increased unit hydrograph peak four fold and the seasonal distribution of floods had changed with increased risk of summer flooding.

Gregory (1974) has presented data on lag time and peak flows from a small catchment near Exeter. An increase in urban development from 8% to approximately 40% had approximately halved lag time and doubled peak discharges. However, although unit hydrographs had been derived for

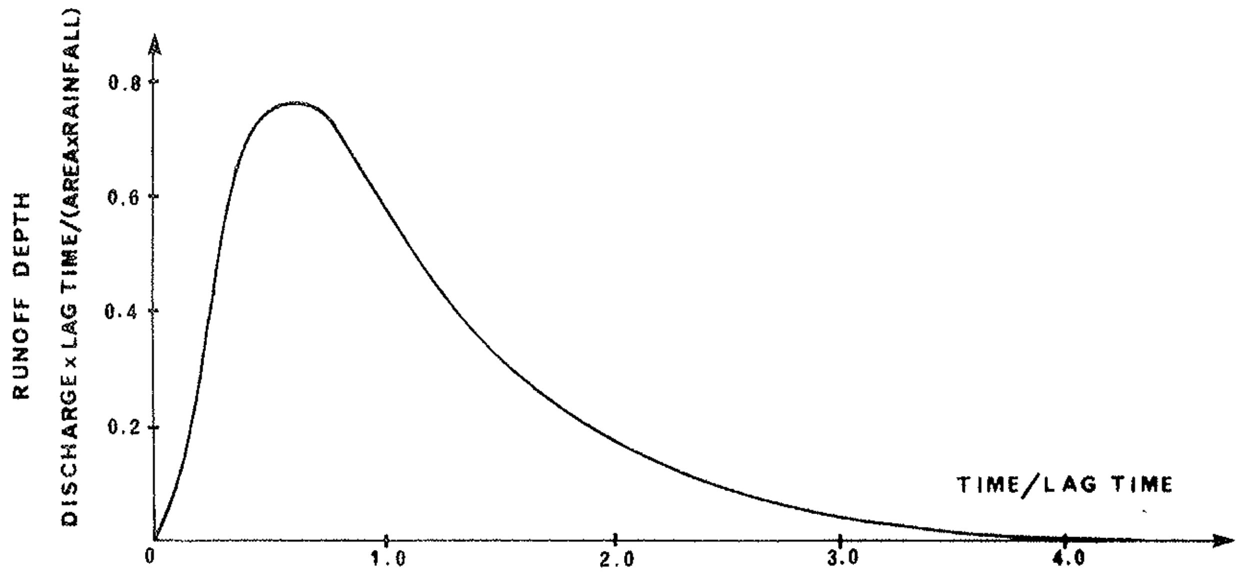


FIGURE 3.8 Non-dimensional unit hydrograph
(from Hall, 1973)

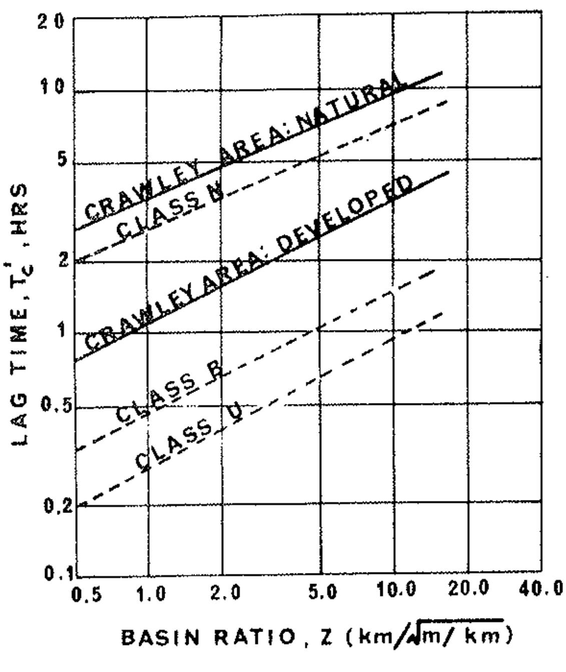


FIGURE 3.9

Lag time against $L/\sqrt{S}$
(from Hall, 1973)

the pre-development conditions, no post-development unit hydrographs were presented.

Schulz and Lopez (1974) derived 5 minute unit hydrographs from 9 small catchments in Denver. They related a number of lag time measures (time to peak, median, centroid, etc) to catchment and storm characteristics, but recommended the following:

$$t'_c = 210.\ D^{.031} R_e^{.304} (A/P)^{.80} (1+I)^{-.342} \tag{3.50}$$

$$q_p = 32.2\ t'^{-.367}_c R^{-.036} \tag{3.51}$$

where D is rainfall duration (hr)

R, R_e are total and effective rainfall depth (in)

A,P are catchment area (mi^2) and perimeter (mi)

and I is the proportion of the catchment impervious

Equation 3.50 predicts only a small reduction in t_c with urbanisation - 10% for I = 35%.

* USDA (1975) have recommended use of the SCS soil-cover-complex method to estimate effective rainfall (see section 3.3.1) and a triangular D hour unit hydrograph given by:

$$t_p = D/2 + t'_c \tag{3.52}$$

$$q_p = 484/t_p \tag{3.53}$$

t'_c for use in equation (3.52) is found from the following equation:

$$t'_c = \frac{L^{0.8}\ (S + 1)^{0.7}}{1900\ S_o^{0.5}} \cdot K_1 \cdot K_2 \tag{3.54}$$

where $S = (1000/CN) - 10$ (3.55)

L is hydraulic length of catchment (ft)

S_o is average land slope (%)

CN is the runoff curve number (see Table 3.3 Section 3.3.1)

and K_1, K_2 are factors to account for impervious areas and channelisation (see Figures 3.10)

No information is given as to what data equation 3.54 and figures 3.10 are based on, but they predict reductions in t'_c due to 35% imperviousness and 100% channelisation varying from about 15% to 75%

* NERC (1975) analysed data from 132 catchments in the UK, of which 24 had urban content greater than 10%. They present a triangular one-hour unit hydrograph defined by

$$q_p = 220/t_p \tag{3.56}$$

$$t_p = 46.6\ L^{0.14} S^{-.38} RSMD^{-.42} (1+URBAN)^{-1.99} \tag{3.57}$$

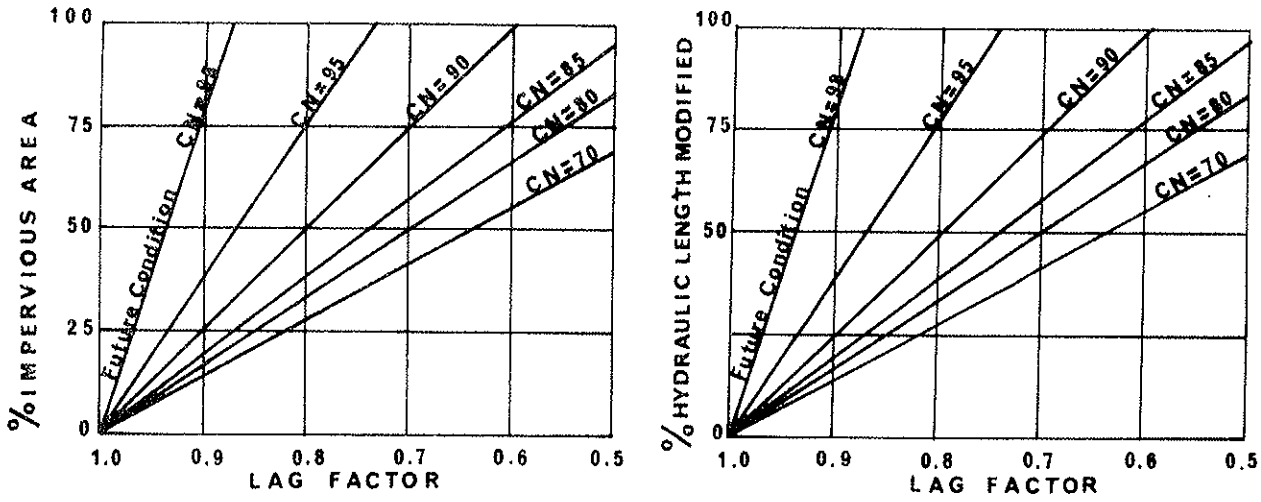


FIGURE 3.10 **Lag factors for various degrees of imperviousness and and channelisation**
(from USDA, 1975)

where q_p is of the peak of the unit hydrograph (m^3/s) due to 1 cm of rain over 100 km^2

t_p is the time to peak of the unit hydrograph (hours)

L is main stream length (km)

S is main stream slope (m/km)

RSMD is the one day net rainfall of 5 year return period

and URBAN is the fraction of the catchment urbanised

Note that the fraction of the catchment urbanised is used in equation (3.57), not the fraction impervious. This is because accurate estimation of imperviousness in large catchments is a lengthy procedure, and most short cuts are subjective. URBAN can be considered equivalent to the fraction of the catchment sewered. Equations (3.56) and (3.57) were developed for use with a constant baseflow addition and a constant proportional rainfall loss. They predict that fully urbanising a catchment will reduce t_p by 75% and increase q_p by 300%.

Okuda (1975) compared unit hydrographs from 3 basins (30%, 50% and 90% urbanised) in Nagoya, Japan. He found that the most urbanised catchment had the shortest response time, and the least urbanised the longest, but he did not try to generalise his results to account for differences in catchment size, length, slope, etc. Ikuse *et al* (1975) present unit hydrographs for two small catchments in Tama New Town, Tokyo, but again they do not try to generalise their results. Ichikawa (1975) derived unit hydrographs for pre- and post-urban development in the Oguri River, Tama. He found significant increases in runoff coefficient, and peak flows.

Yoshino (1975) developed lag time relationships based on 16 urban catchments and 60 rural catchments throughout Japan. He presents the following equation:

$$t'_c = C_t \, (L/\sqrt{S})^{0 \cdot 7}$$

where t'_c is lag time from centroid of rainfall to centroid of runoff (hr)
L is channel length (m)
S is channel slope (dimensionless ratio)

and C_t is 2.44×10^{-4} for urban catchments
or 1.67×10^{-3} for rural catchments.

Use of these C_t values predicts an 85% reduction in lag time with urbanisation.

This summary of the effect of urbanisation on unit hydrograph characteristics has shown, again, a wide variation in the effect of urbanisation. Perhaps, on average, urbanisation might be expected to reduce lag time by about 75% and increase peak by about 300%. The difficulty of separating consistent urban effects from those due to other effects such as non-linearity or time variance has lead to a number of workers using simulation techniques. These are discussed in the next section.

3.3.5 Simulation Models

A. *General Introduction*

Simulation models differ from the unit hydrograph methods of section 3.3.4 in that they attempt to model more closely the various physical processes involved and also the linkages between them. The distinction is by no means precise, and the combination of unit hydrograph routing procedures with particular models for effective rainfall and baseflow would constitute a simple simulation model. The term "simulation" however is usually reserved for models which specify the various catchment response processes in parametric form based on the physical "laws" or analogues thereof. Such models are necessarily complex, and it is only with the continued development of electronic computers of great data handling capacity, speed and accuracy that use of these rainfall-runoff models has become practical. Whereas the limit of the complexity of a model had been the ease with which it could be understood and handled by the user, now all the user need know is the basic details of how the model works, leaving the internal workings to the model builder and the data manipulation to the computer. Improvements in catchment representation can now be made in one or more of the following three ways.

Firstly, the assumptions of superposition and proportionality of linear systems theory may be relaxed, and either non-linear systems theory applied, or kinematic or dynamic solutions sought for the hydraulic equations of non-steady flow. Solution algorithms may thus be chosen for accuracy of description rather than ease of computation.

Secondly, linked system, structure-imitating models can be developed, where each of the various catchment processes is modelled as a separate sub-system. The sub-systems are then linked together in such a manner as to preserve the observed interaction between them and so that they conform to an overall concept of catchment behaviour.

Thirdly, catchments may be divided into subcatchments of more uniform characteristics, and each subcatchment modelled separately. The various subcatchment outflow hydrographs can then be routed to the main catchment outlet. Rainfall-runoff response may thus be monitored at several points in the catchment, and the effects of each subcatchment's outflow on the whole determined.

Simulation models may be classified as isolated-event or continuous models. Isolated event models simulate catchment response to specific storm rainfalls - observed or design events. The problem thus remains of choosing storm and antecedent wetness to give a flood of the required probability. Continuous simulation models, however, are designed to operate throughout long periods, simulating catchment wetness and response during high and low flows. Used with long rainfall and climate records they yield complete synthetic flow records suitable for flood frequency analysis, and thus no link between rainfall and runoff probability is implied. However, continuous accounting of catchment wetness, and synthesis of a large number of flood events requires more meteorological data and more computer time than for an event based model of similar sophistication in its representation of catchment processes.

Simulation models describe catchment response in terms of a set of parameters, each corresponding to a specific subsystem. Models are fitted to the catchment by finding values for the various model parameters. Some may be estimated directly from catchment characteristics (eg area, channel length and slope, percentage imperviousness) but others may need to be found by calibration against observed rainfall-runoff records. Where rainfall-runoff records are not available, values may have to be transposed from nearby gauged catchments or estimated by 'default'; in general the fewer of such "guestimates" necessary the more reliable will be the model performance.

Once fitted, the model will still only yield an estimate of the real catchment response, errors being introduced by an inappropriate or oversimplified choice of model (model bias) and an inappropriate parameter set (parameter bias). Parameter bias may arise from systematic errors in the data used for fitting, or from poor model design or fitting methods with errors in some parameter values being compensated for by opposite errors in others. Because of possible bias, the model's ability to predict the effect of watershed changes by intuitive variation of model parameters should be checked against observed data (remembering again that data may be subject to errors). Unfortunately, unlike the unit hydrograph methods discussed in section 3.3.4, in few instances have predicted effects been checked against observed.

Simulation techniques may be considered to have begun with Horton's (1935) separation of the hydrological cycle into its component processes. The urban drainage engineers (Horner and Jens, 1942; Hicks, 1944; Tholin and Keifer, 1960) were the first to try to assemble models for each process into one overall catchment model, but in spite of several simplifications made possible by the reticularity of sewer systems, the calculations remained unwieldy and generally too complex for manual solution. As a result, it was not until Linsley and Crawford (1960) presented the first version of the computer based Stanford Watershed Model that catchment simulation became a viable technique.

The Stanford Watershed Model was conceived as a continuous simulation model. In its latest version, Mark IV (Crawford and Linsley, 1966), it considers four separate contributions to total runoff: (i) impervious area runoff, found as a fixed percentage of rainfall after interception loss; (ii) surface runoff, found as the rainfall excess after the losses due to "Upper Zone Storage" (Interception, Retention and Depression storages) and "Lower Zone Storage" (Infiltration storage); (iii) interflow, found as a varying percentage of infiltration, and (iv) baseflow found by routing seepage from the "Lower Zone Storage" through Groundwater Storage. Infiltration is found using a modified version of the Philip equation (3.12) allowing infiltration capacity to vary linearly over the catchment. Overland flow routing uses a relationship between depth and discharge based on Manning's equation, and channel routing uses Clark's linear channel-linear reservoir analogy (see 3.3.4). In all, 21 parameters are used to describe the rainfall process of which 8 need to be found by calibration.

The Stanford Watershed Model has been modified and improved by a number of workers, hence: the Hydrocomp Simulation Program (Hydrocomp, 1970); the Kentucky Watershed Model (James, 1972); OPSET (Liou, 1970); and the Texas Watershed Model (Claborn and Moore, 1970) amongst others. It also stimulated the development of other catchment simulation models, both event-based and continuous, of varying degrees of complexity, and of varying intended usage - in particular; the MIT model (Harley *et al*, 1970); HEC-1 (US Army Corps of Engineers, 1973); the EPA model, SWMM (Metcalf and Eddy *et al*, 1971); the USGS model (Dawdy *et al*, 1972); and TR20 (USDA, 1969).

B. The effects of urbanisation

Simulation models have been used quite extensively to estimate the effects of urbanisation, and many models have been specifically designed with this in mind, considering response from pervious and impervious areas separately. However, most studies are "one-offs" and results are rarely generalised for application in other catchments. For this reason, and because many studies are based on intuitive parameter variation and synthetic data, few equations of predicted effects of urbanisation are presented in this section. However, most models applied to estimating the effects of urbanisation are considered, and the modelling techniques used are presented. As before, this section is essentially an annotated list, with major papers identified by an asterisk.
Crawford and Linsley (1965) present some results of a study from the Santa Clara Department of Public Works (1965), California. The Stanford Watershed Model was used to simulate the effect of urbanisation on the rainfall runoff response and flood frequency distribution of several small catchments, intuitively varying impervious area and overland flow length. For drainage design purposes, in order to reduce the computation involved, the Stanford Watershed Model was used to obtain a synthetic

channel inflow record only. This was then subjected to a frequency analysis, and maximum channel inflows corresponding to specified frequencies and durations were mapped over the catchment area. Peak discharges at any point were then found by a simple iterative routing technique to find the critical duration of channel inflow.

* James (1965) applied the Stanford Watershed Model to Morrison Creek California, and, by intuitively varying seven of the model parameters, he obtained estimates of the likely effects of urbanisation on flood discharges of specified frequency. He produced graphs of the separate effects of percentage urban area and percentage channel improvement. Comparing the urban and rural simulated records, he further observed that urbanisation caused a quicker response, a greater sensitivity to small rainfall events and an extended flood season. His graphs were subsequently generalised by Rantz (1971) - see figs 3.3.

* Dempsey (1968) fitted the Stanford Watershed Model to the 65 square mile Pond Creek catchment in Kentucky. Twenty years of flow data were available spanning an increase in urban development from 2.3 to 13.3% and an increase in channelisation from 19 to 57%. The model was first fitted to the pre-urban record and run as a control on the post-urban record, showing flood peaks had typically doubled. The model was then fitted to the post-urban record, and using the information on how model parameters had changed, 12 further hypothetical combinations of imperviousness and channelisation were modelled. Figure 3.11 shows the resulting effect of urbanisation and channelisation on the mean annual and 200 year floods. These figures, although not generalised like Figures 3.3 show a marked difference in trend from those of James (1965).

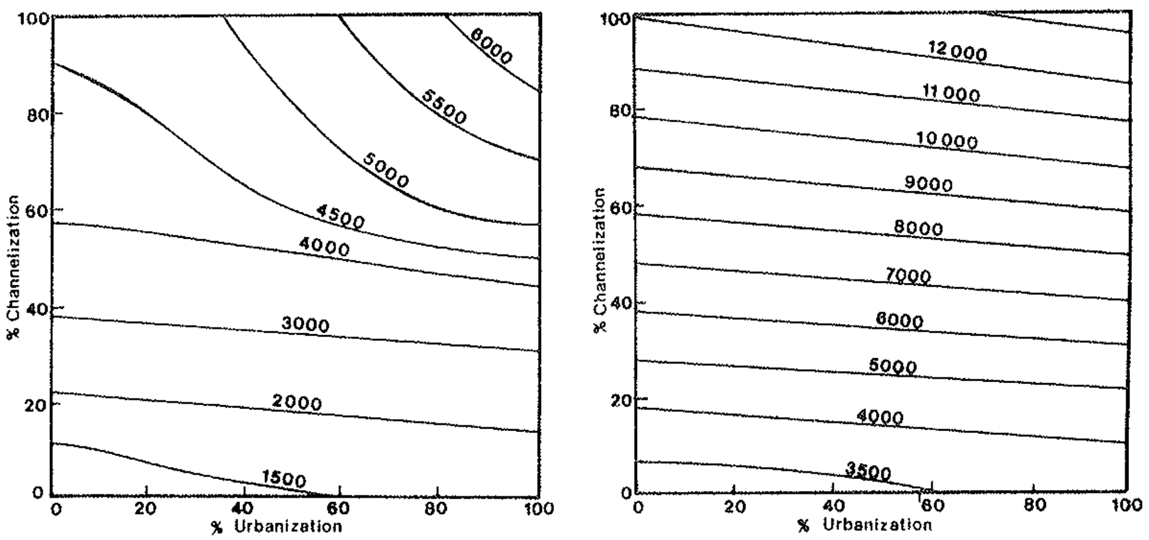


FIGURE 3.11 Effect of urbanisation and channelisation on mean annual and 200 year floods (ft^3/s)
(from Dempsey, 1968)

Other studies using the Stanford Watershed Model or its derivatives are described later in this review - see: Ross (1970); Crawford (1971); Hendricks and Ligon (1973); Ligon and Stafford (1974); Durbin (1974); Barnard and Choley (1976).

Narayana and Riley (1968) developed an analogue computer model for the urbanising Waller Creek catchment in Texas. In the model, one parameter governed the exponential reduction in interception with mass precipitation, two parameters governed the Hortonian infiltration rate (decay constant found from soil properties), one parameter governed the exponential reduction in depression storage with mass net rainfall, and one parameter governed the linear reservoir surface routing. The model was calibrated successively on single years of rainfall-runoff data spanning a period of urbanisation. Regression equations were then found for the five model parameters in terms of percentage impervious cover and a "characteristic impervious length factor" based on the mean flow distance of the impervious area from the design point. The form of the equations did not however permit extrapolation to other catchments. The model was later extended to account for subcatchments (Evelyn *et al*, 1970; Narayana *et al*, 1971) using a linear reservoir for channel routing. Rainfall loss parameters were estimated using the previously derived regression equations (Narayana and Riley, 1968) but subcatchment lag time was estimated using Espey *et al's* (1965) unit hydrograph time to peak equation (see section 3.3.3). Channel delay time was found from Manning's equation combined with a typical average discharge. Narayana *et al* (1971) used the model to generate synthetic data which they then used to calibrate regression equations for flood peaks and volumes in terms of 8 catchment and storm parameters. (Shih *et al* 1975) have also used the model as is described later in this report.

Kinosita and Sonda (1969) modelled the effects of urbanisation on flood runoff in the Syakuzii Catchment, Japan, using both a "macroscopic" approach based on Sugawara's (1961) tank model (a development of the linear reservoir model), and a "microscopic" subwatershed model based on kinematic overland flow routing (using Manning's equation), and kinematic channel routing (using observed stage-discharge relationships). The tank model was fitted to the largest pre- and post-urbanisation storms, showing a threefold increase in storm flow, a reduction in flood plain storage, but no change in baseflow. The kinematic model was fitted to two observed storms on the post urbanisation catchment using a runoff coefficient of .9 for the impervious area and an observed value of .218 for the pervious area. Since reasonable agreement was obtained, the model was used to estimate the discharge that would be expected from a 100% impervious catchment (i) allowing for overbank storage, and (ii) ignoring overbank storage.

Hollis (1970) developed a simulation model of a catchment in Harlow, Essex. The model is continuous giving daily flow volumes. Input consisted of the daily rainfall and monthly Penman evaporation estimates (which were distributed daily using a ratio method). Five separate classes of land use were considered together with a constant riparian area. Interception and depression losses were abstracted as a constant ratio of daily rainfall until satisfied. Effective rainfall was determined as the excess over evaporation and percolation from the soil moisture store. Baseflow was determined as the linear outflow from a groundwater store. The model was basically to investigate seasonality and the water balance, and no channel routing was included. The model was fitted to the pre-urban record and used as a control to identify urban effects in the post-urban record. Hollis found little change in winter runoff, but increases in summer runoff and percentage runoff. Furthermore, increases in percentage runoff were approximately equal to the increase in catchment imperviousness. Overall, increasing imperviousness to 25% had increased the once a year and twice a year daily flows by 300% and 900% respectively.

Ross (1970) fitted OPSET, a self calibrating version of the Kentucky Watershed Model (which is itself a development of the Stanford Watershed Model) to data from 20 catchments, two of which were subject to urbanisation effects. Automatic calibration was used to remove subjectivity in parameter estimation, and thus permit regression of parameter values on catchment characteristics. OPSET was fitted to both the urbanising catchments in a "before-and-after" study, and 7 of the model parameters were regressed on percentage impervious area. The regression equations for the decreases in upper and lower zone soil storage capacity were significant at the 95% and 75% levels respectively, for the flattening of the baseflow recession was significant at the 99% level, and for decreases in evapotranspiration loss and seasonal variation in infiltration rate were just not significant at the 75% level. James (1972) reviews Ross's (1970) work and encourages other workers to apply OPSET to allow generalisation of the results. A later study by Hendricks and Ligon (1973) however found OPSET not sensitive enough to identify effects of urbanisation.

Crawford (1971) applied the Hydrocomp Simulation Program to four small urban catchments to investigate the effects of watershed characteristics on the simulation results. He found that the data necessary for continuous simulation was not readily available - data relating only to events were available, abstractions and return flows were not known, and catchment data had not been collected. Following Brater (1968) he considered effective impervious area as that yielding runoff from small events. He showed HSP could simulate small urban catchments quite well when the data for calibration was available. However, no generalisation of the results for ungauged catchments was possible.

Reimer and Franzini (1971) developed URBDRACONS (Urbanisation's Drainage Consequences), a relatively simple event based unit hydrograph model for use in basin planning. The model was in three sections: sub-basin hydrology; channel routing; and costings for channel or flood plain improvement. Sub-basin hydrology followed the USDA Soil Conservation Service procedures (1957). Effective rainfall was found using US Weather Bureau storm depths with the SCS soil-cover-complex method, and storm profiles were estimated from standard SCS profiles. A triangular unit hydrograph was used for sub-basin routing with time to peak defined from Kirpich's overland flow-time equation (3.18) with adjustments to take account of channel improvement. Channel routing followed the SCS "convex" method, with outflow at time $t+\Delta t$ given as a linear combination (dependent on mean flow velocity) of inflow and outflow at time t. The model was applied to Los Coches creek, California to provide synthetic

100-year floods for pre-urban and various configurations of post-urban conditions.

Roesner *et al* (1972) applied the EPA Storm Water Management Model (SWMM) to a 15 km^2 catchment in San Francisco. The EPA model is an event based model designed to simulate quantity and quality of urban runoff in sewer systems and open channels. The catchment is conceptualised into rectangular subcatchments of known dimensions, slope and surface cover. Rainfall losses due to depression storage, infiltration (given by Horton's equation), and depression storage are abstracted and the excess routed over the surface and along gutters into the pipework using kinematic flow equations. Pipe and channel routing also use the kinematic flow equations, but with an allowance for backwater effects. They demonstrated the ability of the model to simulate watershed response to a single storm, and then they used the model to estimate the changes that would occur if an existing 123 ha park were to be converted to high density residential use. The EPA model has since been used quite extensively, but usually to predict the effects of urbanisation, not analyse observed effects.

Hayden *et al* (1972) used the Barr Engineering Company model to estimate the 100-year flood magnitude and flood plain extent for fully developed conditions in an 82 km^2 urbanising catchment in Minnesota. The Barr Engineering Model is based on the approach outlined by Horner and Jens (1942). It accepts a design rainfall profile as input and abstracts losses due to interception, depression storage and infiltration (using standard infiltration curves obtained from considering Horton's equation as applied to the range of soil cover complexes in the catchment). The rainfall excess hyetograph obtained is routed to subwatershed outlets using synthetic overland flow hydrographs, and channel routing is accomplished using standard ASCE techniques (1949). The model was used to compare different development and flood alleviation strategies.

Kadoya (1973) developed a simulation model to estimate the effect of different development patterns on observed flood discharges in a 1.32 km^2 rural catchment in Japan. The model was originally conceived as a surface routing model alone to convert a given rainfall excess hyetograph to a direct runoff hydrograph. The catchment is conceptualised as a series of rectangular subcatchments of specified slope, shape and roughness, over which surface runoff is routed using the kinematic flow equations. The model was then extended to estimate effective rainfall using Horton's equation and interflow using kinematic routing based on Darcy's Law. He fitted the model to both the rural catchment and a 2.66 km^2 urban catchment, and by transposing surface roughness and rainfall-runoff relationships was able to estimate post-urbanisation flood discharges. He went on to show that using the Rational Method assumptions together with a semi-empirical relationship between time of concentration and rainfall intensity (based on kinematic wave theory) gave essentially the same peak flow values as the solution of the simulation model.

* Leclerc and Schaake (1973) proposed the use of "Stochastic-Deterministic" simulation to generate pre- and post-urbanisation flood frequency distributions. They used a stochastic rainfall generator to yield

synthetic rainfall depths and profiles and input these to the MIT Catchment Model. The MIT model conceptualises the catchment into an assemblage of rectangular subcatchment "modules", each of which consists of a series of overland and channel flow segments to represent the progression of flow from roofs, lawns, sidewalks and streets to gutters and pipes. The modules are connected using Y-branch or reservoir segments. Routing, overland and in channel, uses the kinematic wave equations based on Manning's equation, and rainfall losses are considered using Horton's equation. The model was used to investigate the effect of detailed versus simplified catchment conceptualisation, and a simplified model was selected for the study of a 23 acre catchment in Baltimore. A 200 year stochastic rainfall record was generated and screened to select 48 storms for input to the deterministic model. Flood peaks and volumes-over-thresholds were then subjected to frequency analysis. Stochastic-Deterministic simulation was also applied to a hypothetical 10-acre catchment to demonstrate its ability to derive pre- and post-urbanisation flood frequency curves for various alternative runoff control strategies. The MIT model has also been used by Bras and Perkins (1975) and Wood and Harley (1975).

Hendricks and Ligon (1973) fitted OPSET to 27 years of data from an urbanising watershed in N. Carolina. They also fitted OPSET to a nearby rural control catchment which indicated that the values selected for three land phase parameters (upper zone storage, seasonal variation in upper zone storage, and the evapotranspiration factor) were dependent on annual rainfall. However, in the urbanising catchment, besides an increase in urban fraction from 23% to 56%, no significant trend in land phase parameters was observed, though there was a tendency towards an improved fit with urbanisation. They concluded the adjustment rules for OPSET clouded the effects of urbanisation and that OPSET was not sensitive enough to detect the changes. Ligon and Stafford (1974) extended Hendrick's and Ligon's (1973) work, fitting the Kentucky Watershed Model to the same two catchments, but, instead of using OPSET, they calibrated the model by evaluating and mapping an objective function over a grid of possible parameter values. Four land phase parameters (basic maximum infiltration rate, seasonal infiltration adjustment factor, lower zone storage, and evapotranspiration factor) were optimised to identify the effects of urbanisation, the others were given default values as obtained by Hendricks and Ligon (1973). The optimised parameter values were regressed on land use as determined from aerial photographs, but only the basic maximum infiltration rate showed any significant trend with impervious cover or percentage residential development.

* Durbin (1974) fitted the Stanford Watershed Model to five urbanising catchments in the Santa Anna Valley, California, ranging in area from 9.7 to 216 km^2 and with effective (directly connected) impervious areas from 1% to 31%. He used Crawford's (1971) method to obtain effective impervious area, equating it to the percentage runoff from small events, and presented a graph of effective against total impervious area for the catchments studied. Having calibrated the model for each catchment for both pre- and post-urban conditions he concluded that, within the limits of accuracy of simulation, changes in streamflow with urbanisation could be studied by varying just one parameter, effective impervious area. He then used the model to generate an average of 20 years of data for each basin

for four levels of effective imperviousness: 5, 10, 20 and 30%. The flood frequency distributions for each catchment and condition were derived separately and then generalised in terms of catchment area to produce a set of regional flood frequency curves for each impervious condition (see Figure 3.12). This figure predicts a four fold increase in 2 year flood but only a 1.4 fold increase in 50 year flood as percentage imperviousness increases from 5 to 30%.

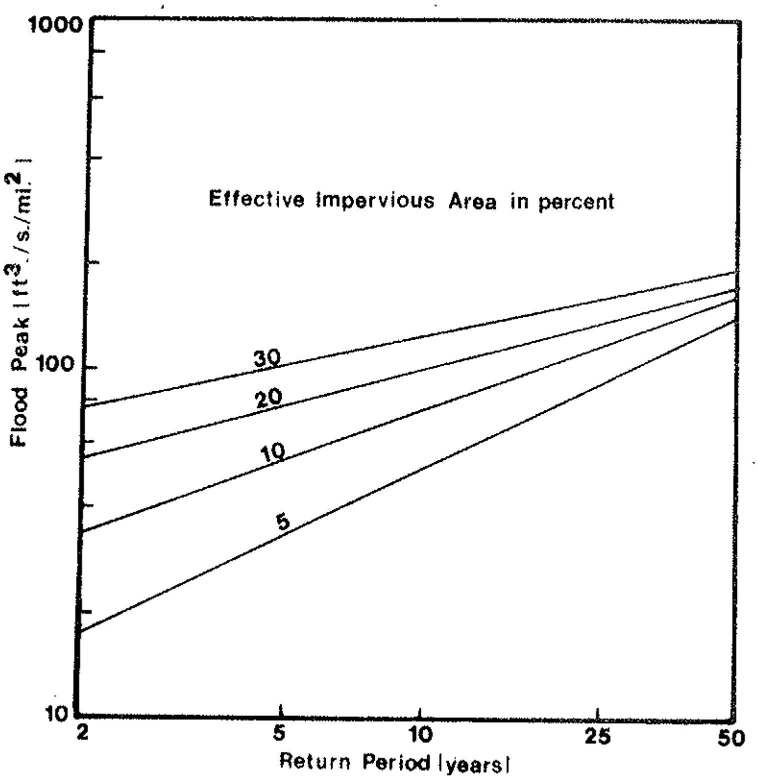


FIGURE 3.12

Effect of impervious area on T-year flood
(from Durbin, 1974)

* Dempster (1974) fitted the USGS Model to an average of 19 storm events on 6 catchments in various stages of urbanisation in Dallas, Texas. The USGS model was modified to handle pervious and impervious area runoff separately. The pervious area component consisted of: (i) an antecedent soil-moisture accounting procedure; (ii) an infiltration procedure, similar to that used in the Stanford Watershed Model, based on Philip's equation (3.12) but allowing variation in infiltration capacity over the catchment area; and (iii) a routing procedure based on Clark's unit hydrograph method (see section 3.3.4). The impervious area component considered only retention storage and routed the excess, also by Clark's unit hydrograph method, but using a different time-area diagram. The model was fitted automatically using the Rosenbrock (1960) optimisation technique. Fitted model parameters were transposed from 6 catchments to a further 8 catchments and the goodness of fit was investigated using an average of 12 storms in each catchment. Since the fit was satisfactory a 57 year observed rainfall record was used to generate a synthetic annual maximum flood series in each of the 14 catchments. The Log-Pearson III distribution was fitted to each series and the 1.25, 2, 5, 10, 25,

50 and 100 year flood values so obtained were regressed on catchment characteristics. The equations he derived have already been presented (equations 3.9) but they yield only small increases in flood estimates, 31% and 15% for 2 and 100 year floods as percentage imperviousness increases from 0 to 35%. The USGS model has since been used by others (Wibben, 1976; Thomas and Corley, 1977; Golden, 1977) to identify the effects of urbanisation, but generally only small increases in flood peaks were found. More recently, Dawdy *et al* (1978) have modified the model to represent the catchment as an assemblage of rectangular subcatchments, using kinematic overland and channel routing.

Lanyon and Jackson (1974) developed the Chicago Flow Simulation Program, a model designed for use at the planning stage, requiring only simple catchment information. To apply the model, the catchment is divided into subcatchments such that reach lengths are essentially straight. Open channel and sewered reaches are considered separately. In open channel reaches, 100% runoff is assumed for impervious areas while infiltration losses are abstracted for pervious areas. Infiltration is assumed proportional to rainfall, average overland flow length, and the inverse of soil moisture content. Soil moisture content is updated during the storm by considering rainfall input less losses due to percolation and evapotranspiration. Overland flow routing is by linear reservoir, and channel routing uses the kinematic equations based on the Manning formula. Overbank storage is considered "off line". In sewered reaches, 100% runoff is assumed from impervious areas and 30% from pervious areas. Overland routing is simplified by assuming a simple time offset with all water stored on the basin at time t becoming sewer inflow by time t+1. Sewer routing uses a linear approximation to Manning's formula. Subcatchment outflows may be controlled to a predetermined maximum, and the necessary flood storage volumes obtained. Simulated and observed hydrographs were compared at four gauges in the 264 km^2 Chicago River Catchment, and peak discharges were presented for six other catchments ranging in size from 12 to 63 km^2. They concluded that the model fell well within the range of the accuracy required for the planning and design of flood control works in urbanising catchments.

Chien and Saigal (1974) present the linearized subhydrograph model, a simple design model for manual or computer based solution. The catchment is divided into subcatchments and a triangular unit hydrograph obtained for each subcatchment based on the "Rational" assumptions and using a time of concentration based on a one dimensional kinematic wave equation. Rainfall excess separation uses separate runoff coefficients, variable in time, for pervious and impervious areas. Channel routing is by the time offset method. They applied the model to a 5.2 ha catchment in Chicago and concluded it was a useful tool, simple to apply.

Shih *et al* (1975) fitted a simplified version of Narayana *et al's* (1971) model to 200 storm events on 30 catchments in Texas. The model abstracted losses from rainfall using a constant loss + exponential decay and used two unequal linear reservoirs for surface and channel routing. The model was fitted automatically and model parameters regressed on catchment characteristics (equations not given). A complementary regression study of rainfall losses and storm profiles yielded statistical distributions of inputs to the model. Sampling from these inputs the model was applied over a range of catchment characteristics, and "concurrency charts" were drawn up from the overlapping probabilities of peak flow values obtained. The charts allowed the estimation of peak runoff rates of specified recurrence interval for small ungauged urban watersheds.

Bras and Perkins (1975) applied the MIT model to a hypothetical 4.7 ha catchment typical of Puerto Rico, assuming both rural and 50% impervious conditions. The effects of urbanisation in Puerto Rico are not generally considered to be great since (i) natural infiltration rates are already low and (ii) houses tend to have flat roofs of large detention storage capacity. However, simulation runs showed average increases in runoff of 100% for 10 year storms and 10% for 50-year storms. Separate computer runs discounting infiltration and roof storage showed that reduced infiltration accounted for about half the increase, while roof storage had no significant effect since the downpipe sizes were not designed to choke the flow. By combining the response of 10 separate 4.7 ha catchments, the effect of urbanisation on larger areas was investigated. With 7 of the 10 subcatchments urbanised, an increase in peak discharge of 70% was obtained, of which 20% was due to channelisation of the main collector stream.

Wood and Harley (1975) applied the MIT stochastic-deterministic catchment model together with a channel flood routing model, a flood damage model and a flood management model in order to investigate the effects of urbanisation and flood control strategies (including the existing flood relief channel) on a catchment in Puerto Rico. They generated two 200 year and one 40 year synthetic rainfall records, and abstracted from each record the largest 40 storms (ranked according to an index of both intensity and duration). To cut down the number of model runs they ran a set of 11 of the most severe storms and fitted a simple peak-flow formula to the resulting floods. This formula was then used with each set of 40 storms to develop the flood frequency distribution. The set of 11 storms were also used to obtain flood stages and hence damage costs, both with and without the existing flood relief channel. A similar analysis to that for flood peaks was used to yield the damage-frequency distribution. It was found that although the flood relief channel routed the main channel away from the urban centre, flood levels were not significantly reduced since the centre was not protected from a fast responding urbanised tributary. The flood management model provided a least-cost solution involving flood control reservoirs and relief channels.

Okuda (1975) considered several simulation models of direct runoff (besides the unit hydrograph model reported in section 3.3.4) in his study of three catchments (30%, 50% and 90% urbanised) near Nagoya, Japan. He gives K and n values for the non-linear reservoir

$$S = Kq^{n} \qquad (3.59)$$

where s is catchment storage

and q is outflow

and values for "equivalent roughness" for use in a distributed kinematic wave model. He also gives values for use in Sugawara's (1961) tank model in which successive parts of the storage-outflow relationship are in effect modelled as separate linear reservoirs. He does not, however, generalise his results to separate urban effects from those of catchment size.

* Aitken (1975) extended Laurenson's (1964) runoff-routing model to urban catchments. The model is a runoff routing model only, and rainfall excess must be estimated separately. The catchment is divided into 10 subareas delineated by equi-spaced isochrones (lines of constant travel time to the outlet). Subarea inflow (equal to rainfall excess over the area plus upstream inflow) is routed through the subarea using a non-linear reservoir, where

$$S = Bq^{1-n} \tag{3.60}$$

S is subarea storage
q is subarea outflow
n is a fixed exponent
and B is a catchment constant

The model was fitted to an average of 6 events from each of 6 urban catchments ranging in size from 1 to 56 km^2 and in urban fraction .25 to 1. Rainfall excess separation was by an initial loss plus continuing loss rate. On the basis of previous work, he used a fixed value for the exponent(n) of discharge in the time delay equation (3.60) of .285 and optimised the value of B to obtain the best estimate of peak discharge. He presented an equation for B in terms of catchment area (A), channel slope (Sc) and fraction urbanised (U).

$$B = 0.285A^{.52}\,(1 + U)^{-1.97}\,Sc^{-.50} \tag{3.61}$$

He then fitted the model to 5 rural catchments and presented a new equation for B based on all 11 catchments.

$$B = 0.581A^{.44}\,(1 + U)^{-2.74}\,Sc^{-.34} \tag{3.62}$$

When applied to observed storms on three catchments, equation (3.62) gave increases in peak discharge for U increasing from 0 to 1.0 ranging from 660% to 1380%. Black and Aitken (1977) have subsequently incorporated the Laurenson routing model described above into the Australian Representative Basin Model in which soil moisture is simulated continuously and an effective rainfall model has been included based on Philip's equation (3.12). Laurenson and Mein (1978) however have developed the original routing model such that subareas are chosen not from isochrones but by dividing the catchment into subcatchments. The relative delay time between subcatchments is defined from channel length (and, in urban catchments, slope), and the overall delay (B) for the whole catchment is defined by optimisation. They present an equation for B in terms of catchment area only, but this is intended only as a first estimate for use in optimisation.

* USDA (1975) used the SCS TR-20 model (USDA, 1969) to obtain approximate tabular and graphical methods of peak flow estimation in urban areas. In the model, the catchment is divided into subcatchments. Design or

observed rainfall profiles may be input, and the rainfall excess distribution is found using the SCS-cover-complex method (see section 3.3.1). Subcatchment routing uses a standard dimensionless unit hydrograph shape which is scaled up using area and time to peak (see section 3.3.4). Channel routing uses the convex method and reservoir routing is by the normal level pool equations with specified outflow control. To derive approximations to the full model subcatchment hydrographs (resulting from a standard profile rainfall excess of unit depth over unit area) were routed to the outlet and tabulated for a range of subcatchment times of concentration and channel travel times. These standard subcatchment hydrographs could then be scaled for area and rainfall excess and superimposed at the design point. A plot of hydrograph peak against time of concentration was also produced to give a peak flow estimate for a single lumped catchment. Finally a simple method was given for the approximate determination of flood storage required to reduce flood flows to a specified limit.

Wittenberg (1976) developed a parallel cascade model to analyse urbanisation effects in the Emscher basin, Germany. 195 events from 4 catchments were analysed spanning a period of 20 years during which urban development increased from 30% to 50%. Noticing time to peak had not been significantly affected by urbanisation, but peak flow and recession rate had, and noticing moreover that lag time increased with percentage runoff when appreciable runoff from pervious area was expected, he proposed separate Nash cascades (see section 3.3.4) for pervious and impervious surfaces. Input to the impervious cascade consisted of total rainfall less an initial loss of 1 mm, while input to the pervious cascade was zero during an initial abstraction period followed thereafter by a constant proportion of rainfall (chosen such that total rainfall excess equalled total direct runoff). Model optimisation gave sensibly constant n and K values for each cascade, and only α, the notional proportion impervious which governs the relative contributions from the faster impervious cascade and the slower pervious cascade, showed any consistent trend with urbanisation. Relating α to the fraction urbanised (U), he presents the equation

$$\alpha = .032\, U^{1.58} \qquad (3.63)$$

He used the model with observed rainfall to predict catchment response under urban conditions, fitting the Pearson III distribution to annual maxima. Barnhard (1977) extended the analysis to 10 catchments and presents equations for K and the product nK for each cascade

$$K_1 = 0.38 + .0037A \qquad n_1K_1 = .2A^{.57} \qquad (3.64)$$

$$K_2 = 3.6A^{.14}\, S^{-.47} \qquad n_2K_2 = 2.0 + 2.1\, n_1K_1$$

where K is in hours
n is dimensionless
A is catchment area (km^2)
and S is catchment slope (m/km)

Schröder (1976) applied a parallel cascade model with 3 cascades to estimate the effects of continuing development in the Niedereschbach catchment, Germany. Rainfall was again separated into impervious and pervious area rainfall according to a ratio, α. Pervious area rainfall was however further allocated to losses (taken as constant plus exponential decay), interflow (taken as constant minus exponential decay) and surface runoff (the remainder). Impervious area runoff and pervious area surface runoff were routed through parallel Nash cascades and then superimposed to give surface runoff, while interflow was routed through a third cascade and superimposed on the groundwater recession (taken as an exponential decay from the flow at the beginning of the event) to form baseflow. Using the model, a growth in impervious area from 18% to 27% yielded an increase of 25% in flood volume. A parallel cascade model has also been used by Diskin (1978).

Jackson *et al* (1976) fitted the US Army Corps of Engineers model, STORM, to seven catchments in the Baltimore-Washington area. STORM is a simple continuous simulation model designed to give estimates of hourly runoff volumes and quality at the planning stage. No explicit flow routing is considered. Runoff volumes are computed as the product of a composite runoff coefficient (the area weighted sum of pervious and impervious area runoff coefficients) and rainfall less detention. In order to avoid problems of large variation in runoff coefficient between storms, and since the model was to be used to estimate annual events, the model was fitted only to large events. Based on their results, they present regional values for depression storage (6.35 mm on pervious area and 1.5 mm on impervious areas) and runoff coefficient (32.4% on pervious areas and 86.2% on impervious areas). These values were then tested by comparing the flood frequency distributions obtained by (i) fitting STORM independently to observed events on the Fourmile Run watershed in Virginia, and (ii) using the regional equations to estimate runoff coefficient and depression storage. The two curves were in close agreement and tied in well with the flood frequency distribution obtained from the historic record. It was thus concluded that STORM could provide adequate estimates of post-urban flood frequency in ungauged catchments.

Saah and Watson (1976) applied the US Army Corps of Engineers model, HEC-1, to two nested catchments in the Santa Clara Valley, California. The main catchment of 25 km^2 was predominantly urban, but the small catchment, 7 km^2 upstream, remained in its rural state. HEC-1 considers a catchment as a series of subcatchments. Rainfall losses may be computed either as an initial plus continuing loss or as a variable proportional loss decreasing exponentially with total loss and rainfall intensity. Catchment routing is by unit hydrograph which may be given as input or computed by Clark's (1945) method (where again the area-time diagram may be given or calculated from a standard non-dimensional form). Channel routing uses the Muskingum method, and reservoir routing uses the level pool equations. If observed data are available, model parameters for loss rates, unit hydrographs and routing procedures may be optimised automatically, using a trial and error algorithm. Saah and Watson fitted the model to the rural subcatchment to optimise pervious area loss rates and to obtain the rural unit hydrograph. To fit the model to urbanised subcatchments, "near zero" loss (5% or 10%) was considered for impervious areas, and times of concentration were determined by applying Izzard's overland flow equation and Manning's equation for gutter flow. The model was used to predict catchment response to the 10 and 100 year rainfall events. The results obtained lay close to and between those obtained by (i) using a regional rural flood-frequency formula and (ii) fitting the Log-Pearson III distribution to the observed non-stationary annual maximum series.

Barnard and Croley (1976) fitted the Iowa Institute of Hydraulic Research model (a modification of the Kentucky version of the Stanford Watershed Model) to the Ralston Creek Watershed in Iowa. The catchment divides into two main branches, each separately gauged: the 7.6 km^2 South Branch, approximately 30% urbanised, and the 7.8 km^2 North Branch, predominantly rural. The model was used to investigate the effect of urbanisation on the flood frequency distribution of the South Branch. The South Branch record did not contain any pre-urbanisation data, so the model was fitted to the North Branch and land surface parameters transposed. Post-urbanisation parameters were estimated by fitting the model to the most recent 3 years of record from the South Branch. The model was then used with an observed record of 33 years of hourly rainfall data to estimate pre- and post-urbanisation daily flow frequency distributions. The results indicated increases in flood levels with urbanisation for floods of less than the 5-year recurrence interval only.

Diskin *et al* (1978) fitted a parallel cascade model to Arcadia watershed in Tucson, Arizona. They considered the same initial loss on pervious and impervious surfaces followed by a continuing constant loss (ϕ) on pervious surfaces only. To reduce further the number of parameters to be optimised, they considered a fixed 2 reservoir cascade for impervious area, and a 3 reservoir cascade for pervious area. Mean values for imperviousness α, and K_1, K_2 and ϕ were estimated using 5 storms, and applied to 6 further storms. Generally good simulations were achieved.

* To summarise this section on simulation techniques for estimating the effects of urbanisation, it is clear that a large number of models have been developed, though most are of a "one-off" nature and relatively few have been extensively used. Consequently there is little hard evidence of the effect of urbanisation on model parameters. Several models, however, stand out as of particular applicability. Those based on unit hydrograph subcatchment routing, like HEC-1 (US Army, 1971), TR-20 (USDA, 1969), the original USGS model (Dawdy *et al*, 1972) and possibly the parallel cascade models (Wittenberg, 1976, *et al*) are readily applied to urbanising catchments because of the extensive experience and data available on unit hydrographs (see section 3.3.4). The Laurenson and Mein (1978) runoff routing model has been quite extensively applied in Australia and is attractive because of its general simplicity in spite of its non-linearity. Of the more complicated models, the Stanford Watershed Model (in its various versions) has been applied the most and shown itself generally capable of accurate prediction of the effects of urbanisation. It does, however, require a lot of data, time and effort to apply.

3.4 Review of Methods - Concluding Remarks

As discussed in section 2, the range of design problems in urbanising catchments requires a range of complexity in design methods. The review of literature in sections 3.2 and 3.3 has considered, effectively, three methods: flood frequency/regional analysis models; simple rainfall-runoff models (especially the unit hydrograph model); and complex simulation models. The range of results reported in the literature might suggest that simpler methods are inadequate for estimating urbanisation effects, and that complex models which take account of "everything" are necessary. Simulation models may indeed be more adaptable and may be fitted to almost any catchment, but their main role should be seen as record extension (as used for example by Depster, 1974). Applied to ungauged catchments they require estimation of several unknown parameters, and estimation of (for example) soil porosity, depth to impermeable layer, initial wetness, etc ... may be more error prone than direct estimation of percentage runoff (for which the hydrologist has a feel for what is reasonable). Simple methods are more readily applied and such features as percentage runoff and lag time are easily identified and provide valuable bench marks for comparison between catchments. Intelligently used, with due regard to the conditions to which they apply, simple methods can give results equally as accurate as the more complex methods, and can be of particular use at the planning stage.

For these reasons, flood frequency and unit hydrograph methods have been developed for particular use in UK conditions. Their development is described in the next section. The approach adopted has been to compare how urban catchments depart from the well established NERC (1975) procedures for rural catchments.

Identified in section 1 as of possible major significance in explaining the variation in the effect of urbanisation between catchments was the effect of location. Simple methods cannot usually account for this effect, and preliminary analysis of a subcatchment model, similar in concept to HEC-1 and TR-20, is also described.

4. ESTIMATING URBANISATION EFFECTS FOR UK FLOOD STUDIES

4.1 Introduction

The UK Flood Studies Report (FSR - NERC, 1975), developed a flood frequency technique and a unit hydrograph rainfall-runoff model for particular application to British conditions. The flood-frequency analysis followed an approach similar to Dalrymple's (1960), deriving a mean annual flood equation (based on 532 catchments each with more than 5 years of data) and 10 separate regional growth curves (based on 420 catchments with an average of 18 years of data). The unit hydrograph study was based on data for 1631 events from 138 catchments, but after rejections due to data errors and the withholding of 6 catchments for test purposes, 1447 events were used to analyse percentage runoff, and 1351 events to analyse unit hydrograph shape.

It was not within the brief of the FSR to investigate the effects of urbanisation. However, 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 in terms of URBAN). For each catchment the derived value of URBAN was based on the most recent 1:63360 map, and this value was considered applicable to the whole period of record. 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 effect of URBAN on the growth curve was not investigated. The remainder of this report describes work carried out since publication of the FSR in which the original data set was re-examined to try and identify more clearly the effects of urbanisation. The analysis was not based directly on observed pre- and post-urbanisation records from the same catchment, but on the typical differences between urban and rural catchments. The methods developed from this analysis were intended for particular use with FSR procedures, though they could equally be applied with other flood estimation techniques.

In the FSR, regression equations were derived for the various flood response 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 spuriously be accounted for by slope (and vice versa), and the regression coefficients may not accurately estimate the true effect of URBAN and slope alone. Consequently, the regression coefficient for URBAN may not in general be used to predict the effects of a change in the value of URBAN. To try and overcome this problem this report compares the form of the equations derived (i) when URBAN was included and then excluded from the independent variables, and (ii) when separate rural and urban subsets were used. This work has suggested some modifications which are discussed in detail in sections 4.2 and 4.3.

The methods of the FSR are essentially lumped, in that no account is taken of spatial distribution of catchment phenomena. As discussed in section 1 of this report, the location of urban development within the catchment can have a significant effect on flood magnitudes. Section 4.4 of this report briefly describes a distributed unit hydrograph model developed from FSR procedures. The model is based on (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. The model is similar in concept to HEC-1 and TR-20 discussed in section 3.3.5 but is based on UK data. Development of the model is continuing.

4.2 Flood-frequency approach

4.2.1 The mean annual flood equation

The FSR regional equation for mean annual flood did not contain the variable URBAN

$$\bar{Q} = \text{RM.AREA}^{.94}\ \text{STMFRQ}^{.27}\ \text{SOIL}^{1.23}\ \text{RSMD}^{1.03}\ (1+\text{LAKE})^{-.85}\ \text{S1085}^{.16} \quad (4.1)$$

where $\bar{Q}$ is mean annual flood (m^3/s)

RM is a regional multiplier (given in fig 4.3 later in this report)
AREA is catchment area (km^2)
STMFRQ is stream frequency (stream junctions/km^2)
SOIL is an index of soil runoff capacity
RSMD is the 5-year return period effective rainfall of 1 day duration (mm)
LAKE is the proportion of the catchment draining through a lake
and S1085 is the main channel slope from 10% to 85% of the channel length upstream from the outlet (m/km)

A summary of how these catchment characteristics are determined is given in Appendix 4 to this report. A fuller consideration is given in FSR I pp 296-312. Data from region 6 (Essex, Lee and Thames) were found to be poorly represented by equation (4.1) and were excluded from its derivation. The reasons for the lack of fit were not clear, but region 6 has both a large number of chalk catchments with virtually no stream network and a large number of urban catchments. The observed range in mean annual flood per unit area was huge - from .004 to 1.67 ($m^3/s/km^2$). A separate equation was derived for region 6.

$$\bar{Q} = .373\ \text{AREA}^{.70}\ \text{STMFRQ}^{.52}\ (1 + \text{URBAN})^{2.5} \quad (4.2)$$

Of the 481 catchments that contributed to equation (4.1) and the 50 catchments that contributed to equation (4.2) only 30 and 14 catchments respectively had urban fractions greater than 10%. Moreover, only 12 and 13 catchments respectively had urban fractions greater than 20%. It is not therefore surprising that URBAN was not a significant variable in equation (4.1).

The effect of adding URBAN to the independent variables in equation (4.1) was considered (see FSR Vol. I p. 340), and the resultant equation may be divided by equation (4.1) to give:

$$\bar{Q}_u/\bar{Q} = .92\ \text{STMFRQ}^{.01}\ \text{SOIL}^{-.02}\ \text{RSMD}^{.02}\ (1+\text{LAKE})^{-.01}\ (1+\text{URBAN})^{.47} \quad (4.3)$$

where $\bar{Q}_u$ is the estimate from the equation including URBAN, and the average value is taken for RM.

Equation (4.3) shows that the interaction between URBAN and the other catchment characteristics is generally small. Substituting typical values of STMFRQ, SOIL, RSMD and LAKE for urban catchments (values of

1.5, 0.4, 30, and 0 respectively) gives

$$\bar{Q}_u/\bar{Q} = 1.01\ (1+\text{URBAN})^{.47} \qquad (4.4)$$

Equation (4.4) suggests that fully urbanising a catchment would increase mean annual flood by just 38%, though it must be borne in mind the exponent .47 has a standard error of .37 and is not statistically significant. The corresponding increases implied by the Essex, Lee and Thames equation (4.2) is 465%. Equation (4.2) however, yields generally a fairly poor estimate of rural mean annual flood. Two catchment characteristics have to do the work of the six characteristics in equation (4.1). The higher exponent for URBAN could be compensation for poor estimation of rural conditions.

In developing equation (4.1) several other forms of mean annual flood equations were derived considering all regions together (FSR I pp 318-328), or considering different subsets of the data chosen by data quality, record length and catchment area. A range of exponents for URBAN of 1.04 to 2.06 was obtained demonstrating the risk of using regression coefficients to estimate the effect of changes in catchment characteristics. However, several equations had coefficients for URBAN in the range 1.8 to 2.0 with standard errors of about .26. In particular, FSR I table 4.10b gives equations involving the same variables as equation (4.1) but with the addition of URBAN raised to exponents of 2.06 and 1.90. A similar analysis to that used above to derive equation (4.4) from (4.1) - comparing the equations obtained when URBAN is included and excluded - gives:

$$\bar{Q}_u/\bar{Q} = .64\ \text{AREA}^{.01}\ \text{STMFRQ}^{-.02}\ \text{SOIL}^{-.03}\ (1+\text{LAKE})^{.03}\ \text{RSMD}^{.08}\ (1+\text{URBAN})^{2.06} \qquad (4.5)$$

Substituting typical values (50, 1.5, .4, 0, 30) gives

$$\bar{Q}_u/\bar{Q} = .9(1+\text{URBAN})^{2.06} \qquad (4.6)$$

which may be reasonably approximated as

$$\bar{Q}_u/\bar{Q} = (1+\text{URBAN})^{1.8} \qquad (4.7)$$

Equation (4.7) is considered a good first approximation to the effect of urbanisation on mean annual flood. However, as discussed in section 1.2, representing the effects of urbanisation by a single factor cannot reproduce the observed variability found in practice (see Appendix 1). Carter (1961) was perhaps the first to argue that the effects of urbanisation on mean annual flood should be related to the separate changes in percentage runoff and lag time. A similar approach has been adopted here. Appendix V describes the derivation of the following equation from the unit hydrograph method given in section 4.3

$$\bar{Q}_u/\bar{Q}_r = (1+\text{URBAN})^{2n}\ \{1 + \frac{I}{100}\ (\frac{PR_i}{PR_r} - 1)\} \qquad (4.8)$$

where n is the rainfall continentality from FSR II p26 (taken as 0.75)
I is the percentage imperviousness (taken as 30% of URBAN area)
PR_i is the percentage runoff from impervious surfaces (taken as 70%)

and PR_r is the original rural percentage runoff, taken as

$$PR_r = 102.4 \ SOIL + 0.28(CWI - 125) \quad (4.9)$$

CWI is an index of typical antecedent catchment wetness, defined from average rainfall according to figure 4.1. The first factor in equation (4.8) gives the effect of change in time to peak, the second factor gives the effect of increased imperviousness. Equation (4.8), with the substitutions given, is presented in graphical form in figure 4.2. This figure predicts that completely urbanising a catchment would typically increase mean annual flood by between 215% and 670% depending on whether the original rural percentage runoff was high (50%) or low (10%). It may also be noted from figure 4.2 that when PR_r is taken as the average value for PR_r obtained from the events used in the FSR unit hydrograph analyses (i.e. 42.6), the predicted increases in mean annual flood for URBAN = 0.5 and 1.0 are 101% and 237% respectively. These values agree closely with the increases predicted by the simple "average factor" equation (4.7) derived directly from mean annual flood data - 106% and 248%. Equation (4.8) thus fits the average situation as well as equation (4.7) but also allows the observed variation between catchments to be related to the typical rural response.

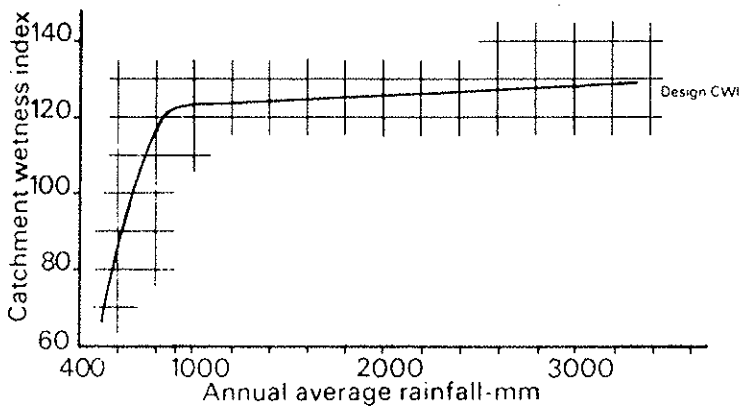


FIGURE 4.1

Design value of CWI

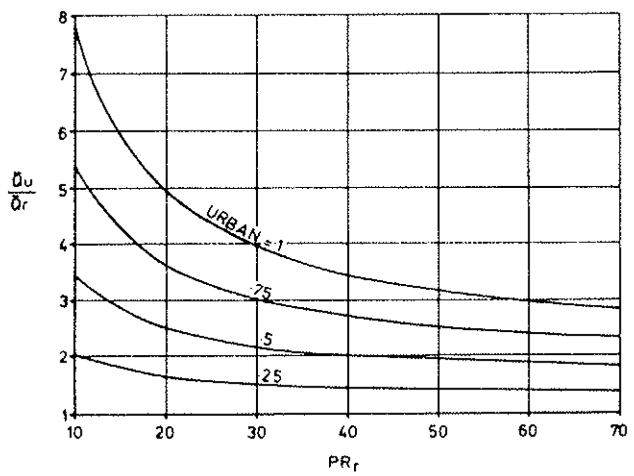


FIGURE 4.2 Ratio of urban to rural mean annual flood against rural condition percentage runoff

To further substantiate that equation (4.8) is a valid estimate of the effects of urbanisation, a regression analysis was performed on the 44 catchments in the FSR data set having urban fractions greater than 0.1. A list of these catchments along with relevant catchment characteristics is given in table 4.1. Writing the right hand side of equation (4.8) as URBF, a regression of $\bar{Q}_u$ was performed on the same variables as used in the FSR regional equation (4.1) but with the additional factor of either (1 + URBAN) or URBF. With either factor the residual square errors of the derived equations were about the same, showing each could be suitably "tuned" to represent the observed data. The optimum exponent for (1 + URBAN) was 1.95, while the optimum exponent for URBF was 0.90 - suggesting that equation (4.8) slightly overestimated the effect of urbanisation. However, in each case, the exponents for the other independent variables were markedly different from equation (4.1), and moreover SOIL, SLOPE and LAKE were not significant at the 5% level. Also, the resultant equations lumped all the catchments together and did not consider regional differences. For these reasons, and to avoid the dichotomy of separate equations for urban and rural catchments, a regression analysis was performed on the ratio $\bar{Q}_u/\bar{Q}_r$, where $\bar{Q}_r$ was estimated from the regional equation (4.1).

As discussed earlier, equation (4.1) was not the recommended $\bar{Q}_r$ equation for region 6 (Essex, Lee and Thames) but the special form (4.2). However, equation (4.2) does not yield a particularly good fit to rural catchments, and consideration was given to using the general equation (4.1). Full details of this analysis are not relevant to this report, but a brief summary follows. Mean annual flood data for the 14 urbanised catchments in region 6 were reduced to rural conditions using equation (4.8). These catchments were then combined with the 36 rural catchments and a best fit multiplier for use in equation (4.1) found (coincidentally, the same value, 0.0153, as for region 5). Optimal regression of the "ruralised" region 6 data still gave significantly different exponents from equation (4.1) (at the 5% level). However, equation (4.1) with the multiplier 0.0153 and urban adjustment by equation (4.8) gave the same standard error as the special form (4.2). Consequently, it is now recommended equation (4.1) should be adopted for estimating $\bar{Q}_r$ in all regions - including region 6.

Regressions of the ratio $\bar{Q}_u$ (observed) to $\bar{Q}_r$ (predicted by equation 4.1 against both (1+URBAN) and URBF (the right hand side of equation (4.8) could now proceed. The analysis may be considered somewhat circular in that URBF was used to define the region 6 multiplier for $\bar{Q}_r$, and then $\bar{Q}_r$ was used to define the optimum exponent for URBF. However, besides the 14 urban catchments, 36 rural catchments contributed to the estimation of the region 6 multiplier; 30 urban catchments from elsewhere were then combined with the 14 from region 6 to investigate the optimum exponent for URBF. Several forms of regression of the ratio $\bar{Q}_u/\bar{Q}_r$ were tried on several subsets of the data. Moreover, regressions were performed on the two terms making up URBF separately. These showed that perhaps more weight should be given to the term (PR_u/PR_r), but overall it was considered that, in the light of the unit hydrograph analyses, the original formu-

TABLE 4.1 Urban catchments used in mean annual flood analysis

Catchment Number	Used in Frequency Analysis	Regional Multiplier	Area (km^2)	Stream Length (km)	Stream Slope (M/Km)	SOIL	Stream Frequency	RSMD	SAAR	CWI	LAKE	URBAN	$\bar{Q}$
19001	-	.0213	369.0	44.6	4.87	.46	.75	36.4	914	122	.04	.11	132.7
19002	✓	.0213	43.8	17.9	5.06	.45	1.07	41.3	1062	123	.00	.14	15.0
19003	✓	.0213	51.8	18.6	7.75	.48	1.27	55.9	980	123	.00	.18	21.4
19006	✓	.0213	107.0	31.7	9.50	.47	1.24	36.1	935	122	.25	.46	38.8
19007	-	.0213	326.0	45.0	8.22	.43	1.25	33.1	891	122	.12	.10	86.1
19802	✓	.0213	137.0	16.1	11.53	.43	1.40	31.0	969	123	.06	.21	52.0
27006	✓	.0213	373.0	44.8	6.43	.36	1.15	39.6	970	123	.30	.13	99.6
27030	-	.0213	311.0	36.6	2.56	.31	.78	30.8	701	103	.00	.11	31.2
27031	-	.0213	245.0	23.7	9.87	.39	1.41	38.1	1107	123	.20	.11	150.6
27846	✓	.0213	1880.0	104.5	1.45	.41	.78	37.4	980	123	.09	.13	343.6
28003	✓	.0213	407.0	27.4	1.45	.33	.30	24.8	742	109	.02	.50	71.9
28004	-	.0213	795.0	35.6	1.34	.42	.43	23.7	734	107	.00	.32	65.6
28005	✓	.0213	1480.0	60.2	1.01	.40	.44	23.4	714	105	.00	.19	100.9
28006	-	.0213	324.0	48.9	1.77	.33	.79	28.8	825	119	.07	.12	28.6
28019	-	.0213	3070.0	88.0	1.03	.39	.49	24.0	731	107	.00	.11	176.6
28032	✓	.0213	63.1	14.7	6.24	.25	.29	30.2	709	104	.00	.17	4.3
28802	✓	.0213	37.8	9.3	8.69	.21	.18	30.4	734	107	.17	.22	9.2
37807	-	.0153	71.4	12.4	2.50	.45	.66	15.6	571	72	.00	.16	6.3
38013	✓	.0153	70.7	14.9	2.01	.15	.13	22.2	666	97	.00	.29	2.7
39003	✓	.0153	176.0	10.4	2.89	.15	.02	25.4	754	110	.00	.32	9.6
39004	✓	.0153	122.0	2.4	4.36	.30	.01	27.1	800	117	.00	.22	2.6
39005	✓	.0153	43.5	7.4	2.28	.33	.20	23.2	640	93	.00	.81	12.8
39012	-	.0153	69.1	9.0	2.55	.30	.19	23.6	691	101	.00	.46	13.0
39820	✓	.0153	25.1	10.8	4.48	.45	.92	23.4	688	101	.00	.50	7.3
39821	✓	.0153	118.0	14.7	3.49	.45	.34	22.7	678	99	.00	.76	22.2
39824	-	.0153	10.3	6.4	11.09	.15	.55	25.1	657	96	.00	.37	5.2
39827	✓	.0153	36.0	8.5	6.73	.15	.61	21.6	661	97	.00	.25	6.2
39830	✓	.0153	10.0	5.3	10.11	.15	.77	24.7	665	97	.00	.60	2.5
39831	✓	.0153	7.0	4.0	16.12	.15	.71	23.8	683	100	.00	.36	2.3
39834	✓	.0153	132.0	25.0	2.32	.45	.53	22.5	667	97	.00	.75	29.2
39840	✓	.0153	29.0	8.7	5.67	.45	.69	23.3	687	101	.00	.46	6.9
42801	✓	.0234	17.4	5.9	7.20	.20	1.38	34.1	803	117	.00	.36	7.6
54004	✓	.0213	264.0	28.8	1.92	.41	.72	22.8	707	104	.00	.15	28.6
54006	✓	.0213	324.0	33.4	2.38	.17	.42	25.1	721	106	.00	.14	19.0
68004	-	.0213	88.1	18.1	4.04	.21	.36	28.4	785	115	.00	.10	10.6
69001	✓	.0213	679.0	70.1	2.52	.45	1.19	40.7	1118	123	.15	.27	174.0
69002	✓	.0213	559.0	57.0	3.83	.48	1.00	41.0	1265	123	.13	.15	231.6
69003	✓	.0213	74.3	16.8	5.26	.49	.58	38.3	1064	123	.00	.58	43.4
69007	✓	.0213	660.0	61.2	3.03	.47	1.22	40.8	1126	123	.16	.45	196.3
69801	-	.0213	145.0	18.8	9.81	.48	1.02	44.8	1303	124	.17	.11	65.2
69804	✓	.0213	150.0	38.2	6.90	.49	.72	46.3	1179	123	.18	.22	89.3
84001	-	.0213	334.0	27.7	12.02	.46	.96	40.9	1245	123	.01	.10	99.8
84012	✓	.0213	235.0	31.3	6.19	.44	1.06	42.2	1264	123	.07	.27	106.4
84803	✓	.0213	130.0	28.8	6.97	.46	.60	40.2	1011	123	.02	.36	30.1

lation of URBF should be retained. In each case URBF explained more of the variance in the data than the simple term (1 + URBAN) alone. Only the overall equation of $\bar{Q}_u/\bar{Q}_r$ through the origin is presented here

$$\bar{Q}_u/\bar{Q}_r = \mathrm{URBF}^{1.06} \qquad (4.10)$$

Thus, analysis of the ratio $\bar{Q}_u/\bar{Q}_r$, unlike the earlier analysis of $\bar{Q}_u$ alone, suggests that URBF fractionally underestimates the effect of urbanisation. When all was considered, the closeness of each exponent to unity was taken as sufficient justification for the use of equation (4.8) - or fig 4.2 - unaltered as an estimate for the effect of urbanisation on mean annual flood. Rural mean annual flood may be estimated using the regional equation (4.1) with regional multipliers as given by fig 4.3.

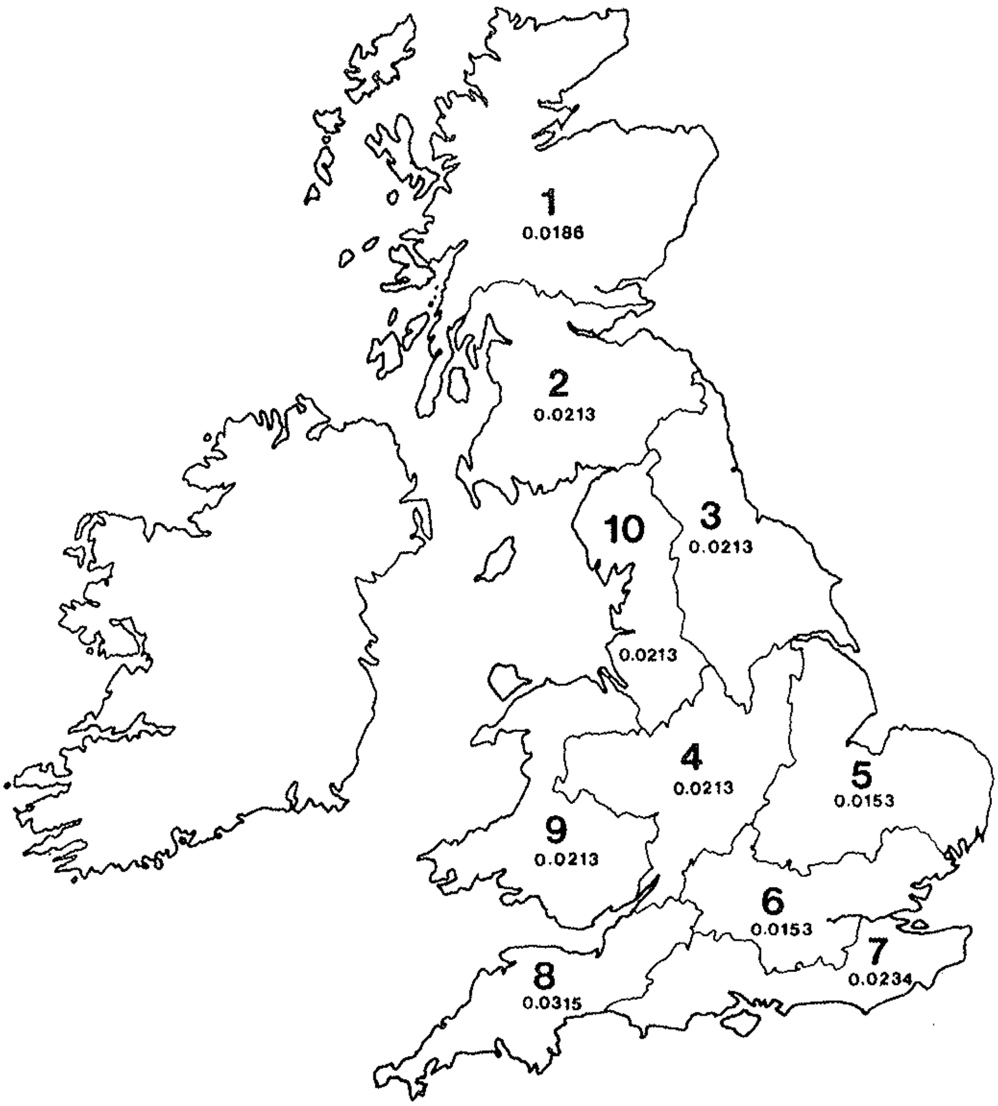


FIGURE 4.3 Region numbers and corresponding multipliers for use in mean annual flood equation (4.1)

4.2.2 The regional growth curve

The previous section has considered the mean of the annual flood distribution; the standard deviation (or more exactly, the coefficient of variance, CV) was also considered in the FSR, but random errors associated with small sample estimates clouded the subsequent regressions on catchment characteristics. Thus, although RSMD, S1085 and STMFRQ were significantly related to CV at the 5% level, they were only able to explain 11% of the observed variance. (see FSR I p 344). Higher moments than CV were not considered since they would be subject to even larger sample errors. By the same reasoning, small sample estimates of T-year floods would also be subject to large errors, and thus separate regression equations for flood peaks of a range of different return periods were not considered. The approach adopted instead was regional analysis.

Each station's annual flood data (mean $\bar{Q}$, standard deviation, s) were standardised, dividing by the mean to yield data with mean=1 and standard deviation = $s/\bar{Q}$ = CV. The earlier analysis had shown CV was subject to large random error, but that some consistent trend with RSMD (in particular), S1085 and STMFRQ was present. Considering this trend to derive from a regional relationship as opposed to a parametric relationship with catchment characteristics, CV may be taken as constant within some suitably defined region. The standardised data may then be combined to yield a regional average estimate of CV. This approach need not however be restricted to CV but may be extended to skewness, or more particularly to the complete flood frequency distribution. In this case, not just CV but the complete frequency distribution of $Q/\bar{Q}$ is considered fixed within the region. Individual catchment data on the frequency (1/T years) with which certain $Q/\bar{Q}$ values are exceeded may thus be pooled to yield a regional average $Q/\bar{Q}$ against 1/T curve. A full discussion of the procedure is given in FSR I pp 170-185. Gumbel reduced variate, y, is used in place of 1/T to linearise the $Q/\bar{Q}$ plot, where y is defined as

$$y = -\ln(-\ln(1 - 1/T)) \tag{4.11}$$

(Tables of y for use with various sample sizes are given in FSR I pp 82-84.).

Catchment data are then pooled by averaging all y values and corresponding $Q/\bar{Q}$ values falling within successive ranges of y (an example follows in table 4.2).

As discussed above, the region curve derivation assumes the same $Q/\bar{Q}$ versus 1/T distribution for all catchments within the region. Urbanisation, however, is generally considered to increase more frequent floods by a greater proportion than rarer floods - thus reducing the observed CV and flattening the growth curve. With the small number of urban catchments available for each region it was not possible to consider separate regional growth curves for urban and rural catchments, so for the present study two approaches were adopted. Firstly, fitted parameter values for the General Extreme Value distribution to individual urban catchments were compared with average values applicable to the regional curve (see FSR I p173). This showed, indeed, an overall tendency to flattening of the growth curve with increasing urbanisation. Secondly, because the above analysis was open to large sampling errors, pooled growth curves were derived for urban catchments, considering all catchments together, irrespective of region.

TABLE 4.2 Pooled urban growth curves (URBAN = .75 and .50)

CATCHMENT -REGION	AREA	URBAN	RANGE OF y WITH NUMBER OF POINTS IN RANGE (n) AND CORRESPONDING TOTALS OF y and $Q/\bar{Q}$																				
			$0<y<.5$			$.5<y<1.0$			$1.0<y<1.5$			$1.5<y<2.0$			$2.0<y<2.5$			$2.5<y<3.0$			$3.0<y<\infty$		
			n	Σy	$\Sigma Q/\bar{Q}$	n	Σy	$\Sigma Q/\bar{Q}$	n	Σy	$\Sigma Q/\bar{Q}$	n	Σy	$\Sigma Q/\bar{Q}$	n	Σy	$\Sigma Q/\bar{Q}$	n	Σy	$\Sigma Q/\bar{Q}$	n	Σy	$\Sigma Q/\bar{Q}$
39005 - 6	43.6	0.81	2	.43	2.10	1	.85	1.11	1	1.44	1.16							1	2.52	1.63			
39821 - 6	118	0.76	5	1.41	4.95	4	3.02	4.39	3	3.66	3.52	2	3.35	2.70	2	4.47	3.09	1	2.93	1.70	1	3.95	1.84
39834 - 6	132	0.75	1	.23	1.00	1	.59	1.07	1	1.01	1.22	1	1.59	1.25				1	2.66	1.33			
Totals			8	2.07	8.05	6	4.46	6.57	5	6.11	5.90	3	4.94	3.95	2	4.47	3.09	3	8.11	4.66	1	3.95	1.84
Average		.77		.26	1.01		.74	1.10		1.22	1.18		1.65	1.32		2.24	1.55		2.70	1.55		3.95	1.84
19006 - 2	108	.46	2	.43	1.75	1	.85	.91	1	1.44	1.23							1	2.52	1.79			
28003 - 5	407	.50	3	.87	3.04	2	1.68	2.09	1	1.26	1.08	1	1.64	1.12	1	2.17	1.23				1	3.22	1.23
39012 - 6	69.1	.46	1	.26	.91	2	1.41	1.91	1	1.27	1.17	1	1.83	1.24				1	2.88	1.90			
39820 - 6	25.1	.50	3	.87	2.56	2	1.68	2.06	1	1.26	1.10	1	1.64	1.13	1	2.18	1.60				1	3.22	2.23
39830 - 6	10.1	.60	2	.43	1.63	1	.85	.94	1	1.44	1.03							1	2.52	2.26			
39840 - 6	29.0	.46	[illegible]	.61	2.67	2	1.39	2.69	2	2.39	2.82	1	1.71	1.49	1	2.25	1.64				1	3.29	1.94
69003 - 10	74.3	.58	3	.87	2.93	2	1.68	2.29	1	1.26	1.29	1	1.64	1.36	1	2.18	1.43				1	3.22	1.43
Average		.51		.26	.91		.80	1.07		1.29	1.22		1.69	1.27		2.20	1.48		2.7	1.85		3.24	1.71

The urban catchments used in the mean annual flood analysis (Table 4.1) were pooled into three groups depending on the value of URBAN: greater than 0.625; between 0.375 and 0.625; and between 0.125 and 0.375.

Catchments with URBAN less than 0.125 were excluded together with catchments 28004, 37807 and 39824 for which data were not available. The mean URBAN values relevant to each band were 0.77, 0.51 and 0.24 which were considered representative of the values 0.75, 0.50 and 0.25. Annual flood data given in FSR IV were pooled and the growth curve derived for each band of URBAN compared with a weighted average "rural" growth curve (defined from the FSR regional growth curves relevant to the catchments used). The derivation of the pooled growth curves for the two higher URBAN bands is given in Table 4.2 where only y values greater than zero (T>1.58 years) are considered. The small number of catchments in the top two bands makes for poor smoothing, particularly where different catchments contribute to successive ranges of y (Note the largest flood from 39830 has been excluded as an outlier, but still the theoretically impossible result of a higher $Q/\bar{Q}$ ratio at a lower y value has been obtained).

The average curves from Table 4.2 together with the URBAN = 0.25 curve have been plotted on Figure 4.4 and compared with their respective weighted average rural curves. Also shown are manually smoothed curves drawn so as to represent the data while maintaining a sensible progressive trend. The curves are based on relatively short lengths of record, (44, 70 and 300 station-years respectively for URBAN = 0.75, 0.50 and 0.25 respectively) and have thus only been extended to a return period of 50 years. An interesting feature of Figure 4.4 is that, as URBAN has increased to 0.75, the return period of the mean annual flood has reduced from about 2.65 years (y = .75) to about 2 years (y = .37). This accounts for the URBAN curve lying above the rural curve until y = 1.8. If the same return period flood (T=2.65) had been used to

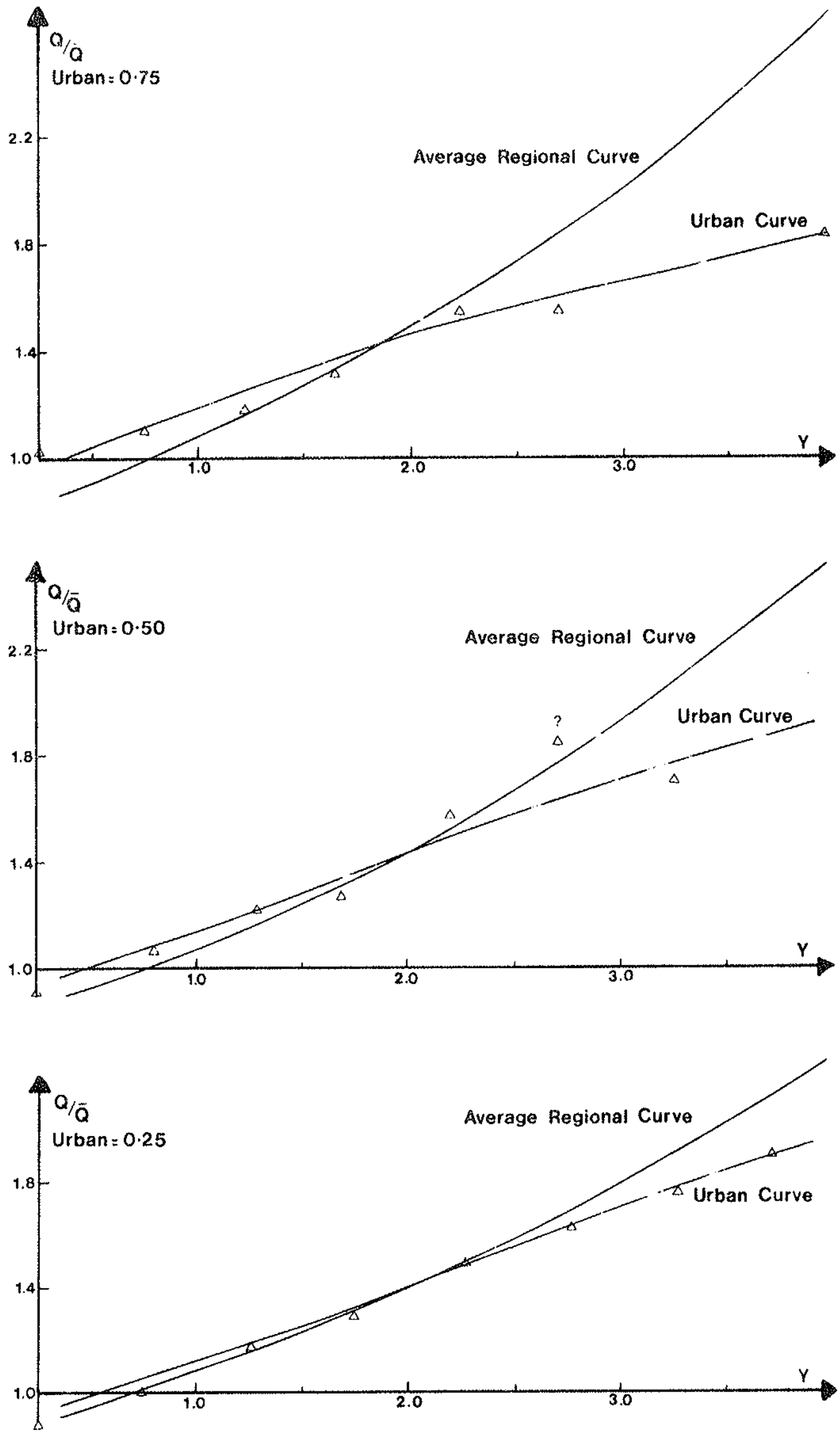


FIGURE 4.4 Growth curves for urban catchments

standardise the urban annual flood data (Q/Q_T) the smoothed URBAN curves would lie wholly below the rural curves.

In order to abstract the information from Figure 4.4 for use with individual regional growth curves an equivalent return period procedure is proposed such that the growth factor for the T-year flood for an urban catchment is found at an equivalent return period T^1 on the rural growth curve. The equivalent return periods have been taken from Figure 4.4 and are presented in Table 4.3 as equivalent y-values where y is defined from T according to equation 4.10.

TABLE 4.3 Equivalent y-values for specified return periods and values of 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 I, Fig 2.14 p 174 to determine growth factors. However, it may be easier to interpolate in Table 4.4.

TABLE 4.4 Regional growth factors 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 4.4, 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 tends 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:

$$Q_{Tu}/Q_{Tr} = 1 + Be^{-ky} \tag{4.11}$$

where Q_T is the T-year flood

y is the Gumbel reduced variate - see equation (4.10)

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

$$k = .48 \{\ln (\frac{\bar{Q}_u}{\bar{Q}_r} - 1) - \ln (\frac{Q_{50u}}{Q_{50r}} - 1)\} \tag{4.12}$$

$$B = (\frac{Q_{50u}}{Q_{50r}} - 1)e^{3.9k} \tag{4.13}$$

Then since y $\simeq$ ln T for large T, equation (4.18) may be rewritten

$$Q_{Tu}/_{Tr} = 1 + BT^{-k} \tag{4.14}$$

These equations (4.12), (4.13) and (4.14) may be used to extend the growth curve beyond 50 years, though it must be stressed that the procedure is largely intuitive.

4.3 Rainfall-runoff modelling

4.3.1 Introduction - the FSR procedure

Of the 132 catchments used in developing the FSR unit hydrograph procedure, 24 had URBAN fractions of 0.1 or more (in the unit hydrograph analysis URBAN was rounded to one decimal place). A full description of the FSR unit hydrograph procedure is given in FSR Vol I ch 6. However, for completeness a brief account is given below. Subsequent work on the effects of urbanisation follow from section 4.3.2 onwards.

In the FSR, for each event 'LAG' was defined as the time from the centroid of total rainfall to a point A (see Fig 4.5), the peak (or in a multi-peaked event the centroid of peaks) of the total runoff hydrograph. Quick response (direct) runoff was then separated from "non-separated flow" (baseflow) by (i) extending the preceding

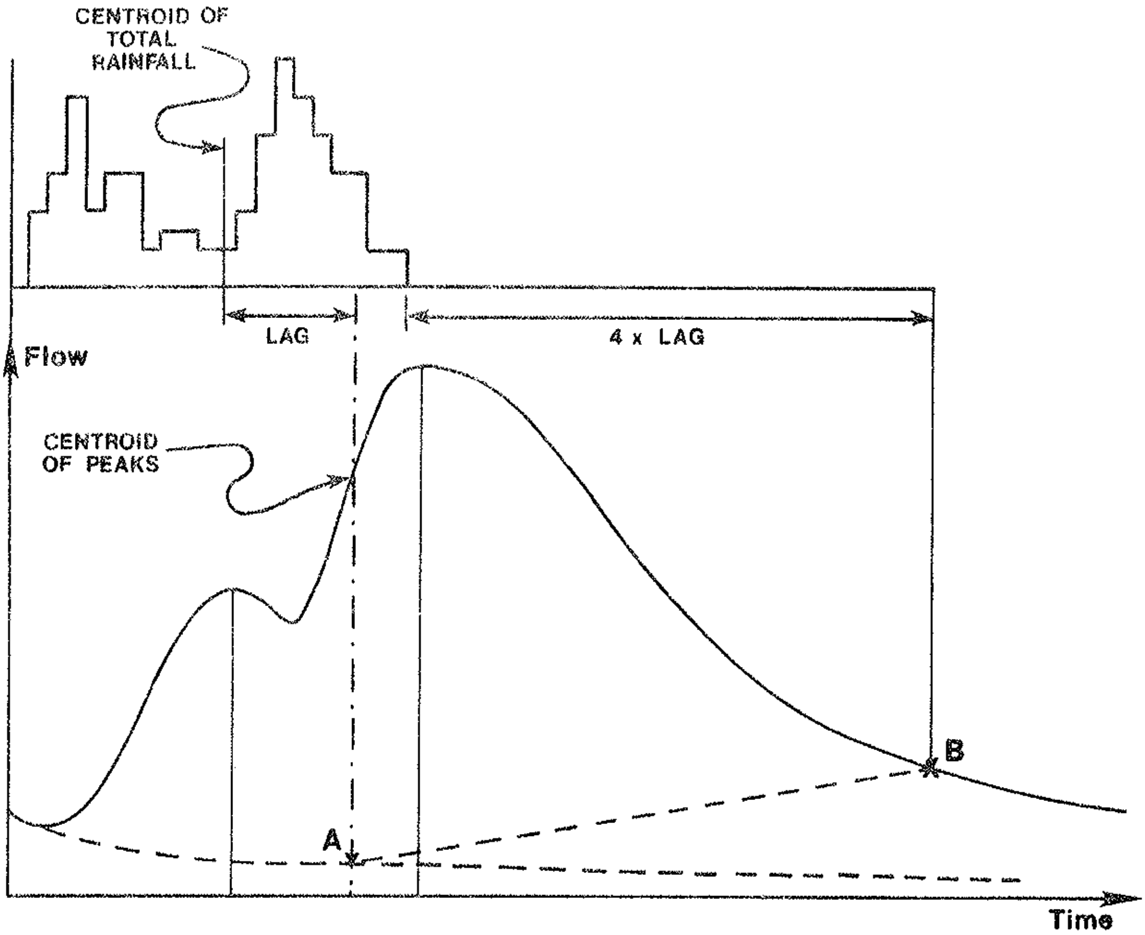


FIGURE 4.5   **FSR method of quick response separation**

(and if necessary the succeeding) recessions to a point B four times the LAG from the end of rainfall, and (ii) joining points A and B with a straight line. Effective rainfall was separated from total rainfall using a variable loss rate technique with loss proportional to (K/CWI) where CWI is the variable Catchment Wetness Index, and K is an arbitrary constant chosen to ensure equal volumes of effective rainfall and quick response runoff. CWI is determined at 9 am on the day of the event from

$$CWI = 125 - SMD + API5 \tag{4.15}$$

where SMD is the soil moisture deficit (mm) estimated nationally by the UK Meteorological Office using a running balance of rainfall less potential evaporation. API5 in the 5 day antecedent precipitation index given by

$$API5 = \sqrt{0.5}\ \{P_{-1} + 0.5P_{-2} + 0.5^2P_{-3} + 0.5^3P_{-4} + 0.5^4P_{-5}\} \tag{4.16}$$

or

$$API5 = \sqrt{0.5}\ \{P_{-1} + \sqrt{0.5}\ API5_{-1}\} \tag{4.17}$$

where P is the daily rainfall total (mm) and the suffix -n refers to day n before the event. The constant 125 in equation (4.15) is to ensure CWI rarely goes negative. To update CWI during the event the daily decay rate (0.5) in equation (4.17) is replaced by $(0.5)^{t/24}$, where t is the relevant time interval; potential evaporation is ignored.

Having separated quick response runoff and effective rainfall, the unit hydrograph was derived by the technique of matrix inversion. Unit hydrograph peak (Qp), time to peak (Tp) and width at half-peak (W) were abstracted and catchment average values of Tp, QpTp and W/Tp were regressed on catchment characteristics to yield

$$Tp = 46.6^{-.38}\ RSMD^{-.42}\ MSL^{.14}\ (1 + URBAN)^{-1.99} \tag{4.18}$$

$$Qp\ Tp = 162 + 2.6\ Tp \tag{4.19}$$

$$W/Tp = 1.40 + .0083\ Tp \tag{4.20}$$

However, following a pilot study, a variable *proportional* loss technique of effective separation was preferred to variable loss *rate*. Percentage runoff was set equal to K.CWI where again K is an arbitrary constant to ensure equal volumes of direct runoff and effective rainfall. As a result of this pilot study the Tp equation (4.18) was unaltered, but equations (4.19) and (4.20) governing unit hydrograph shape were changed to

$$Qp\ Tp = 220 \tag{4.21}$$

$$W/Tp = 1.26 \tag{4.22}$$

which (in the units used) describe a simple triangle with time base equal to 2.525 time to peak. The original equations (4.19) and (4.20) are presented here because the data published in FSR IV refer to loss rate separation and not percentage runoff separation as used in the pilot study.

Finally, the overall percentage runoff for each event was regressed on catchment and storm characteristics to yield:

$$PR = 95.5\ SOIL + 0.28\ (CWI-125) + 0.1(P-10) + 12.0\ URBAN \tag{4.23}$$

where SOIL is an index of soil runoff potential
CWI is the catchment wetness index at the start of the event
and P is storm rainfall (mm)

and the Average Non Separated Flow, ANSF($m^3/s/km^2$), over the duration of quick response runoff was regressed on catchment and storm characteristics to yield

$$ANSF = .001\ \{.326\ (CWI-125) + .75\ RSMD + 3.0\} \tag{4.24}$$

where RSMD is the 5-year return period effective rainfall (mm) of 1 day duration.

In order to apply the above procedure in design, a *constant* proportional loss separation of effective rainfall was recommended, using the overall percentage runoff given by equation (4.23). Also a constant ANSF was recommended as given by equation (4.24). It remained to choose a rainfall depth duration and profile, and a value for the antecedent condition index CWI. To this end a simulation exercise (FSR I ch 6.7) was performed to derive recommendations for these inputs that would consistently yield flood peaks that matched the observed flood frequency distribution.

4.3.2 Effective rainfall separation

The variable loss rate technique of rainfall separation used in the FSR was an objective technique in that it was applied consistently to all events. It was meant to be a realistic technique without trying to represent the physical processes involved. No optimisation of the technique (for example by raising CWI to some power) was tried. In these circumstances, although an alternative technique (or an alternative definition for CWI) might be better suited to urban catchments, it was not considered necessary to investigate the possibility - any major differences should appear in variation of the derived unit hydrograph shape.

While the effect of urbanisation on loss distribution has not been considered, it was expected that urbanisation would have a significant effect on overall loss volume. The FSR percentage runoff equation (4.23) does indeed imply that totally urbanising a catchment would increase percentage runoff by an addition of 12%. However, regression coefficients should not be used to estimate the effect of change in catchment characteristics. Moreover, equation (4.23) makes no allowance for whether the urban area replaces soil surfaces of high or low infiltration capacity. To investigate the significance of this, a new percentage runoff equation was derived using only the rural catchments from the FSR data set (FSR IV).

$$PR_r = 102.4\ SOIL + 0.28\ (CWI - 125) + 0.10(P-10) - 1.9 \qquad (4.25)$$

Full details of the regression are given in Table 4.5 below

TABLE 4.5 Regression for percentage runoff, rural catchments

Variable Name	Coeff	Standard Error	t Statis.	R^2	Standard Error of Est.	Constant
No. of observations - 1074						
SOIL	102.37	5.82	17.6	.39	15.4	- 1.9
(CWI-125)	.28	.02	12.8	-	-	-
(P 10)	.10	.02	5.3	-	-	-

The coefficients of SOIL and CWI differ by about 1.5 and 3 standard errors respectively from the original FSR equation (4.23), suggesting fundamental differences between the urban and rural catchments. Moreover, the change in coefficient values is in the expected direction with soil type and antecedent condition being more significant in rural catchments.

It may also be noted that equation (4.25) has a smaller R^2 value and a higher standard error than the original FSR equation (FSR Vol I p 419), showing that since equation (4.25) is the "best fit" to rural catchments, percentage runoff from urban catchments is generally less variable. Based on these results, a new "intuitive" percentage runoff equation was tried, of form similar to that proposed by Snyder (1958) and Carter (1961).

$$PR = PR_r \frac{100 - I}{100} + PR_i \frac{I}{100} \qquad (4.26)$$

where PR_r is the rural percentage runoff from equation (4.25)

PR_i is the impervious area percentage runoff

and I is the catchment overall percentage imperviousness.

The percentage imperviousness in any catchment will depend not only on the degree or urbanisation but also on the type of development (city centre, industrial, residential). Typical impervious fractions for a range of development types were considered in section 3.3.1, but surveys have shown a good average value for catchments greater than about 2 km^2 may be taken as, I = 30 URBAN (ie 100% urbanised ≡ 30% impervious). With this relationship for I and using equation (4.25) for PR_r, equation (4.26) was applied to the FSR data set giving a best fit value for PR_i of 63%. However, the work of Kidd and Lowing (1979) has suggested a better value for PR_i of 70% and this has been adopted. In practice, equation (4.26) is not very sensitive to small changes in the value adopted for PR_i. Substituting the recommended values into equation (4.26) gives

$$PR = \{102.4\ SOIL + 0.28\ (CWI-125) + 0.10\ (P-10) - 1.9\}.(1-0.3\ URBAN) + 70\{0.3\ URBAN\} \qquad (4.27)$$

This equation applied to the FSR data set yields a standard error of estimate of 15.02, which is a slight improvement over the value of 15.09 for the original FSR equation (4.23). 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.

To test further whether equation (4.27) adequately accounted for urbanisation, observed PR values for the urban catchments in the FSR data set were "ruralised" by subtracting the impervious area component (21 URBAN) and scaling the pervious area component up to the full catchment area. An analysis of variance was then performed on the rural and "ruralised-urban" data sets. This showed unfortunately that significant differences still existed between the data sets. Compared to equation (4.25), the ruralised-urban subset gave a SOIL coefficient of 106.4, a CWI coefficient of 0.17 and a P coefficient of zero. Inspite of this somewhat disappointing result, equation (4.27) was still considered to give the best estimate of the effect of urbanisation - until further work can be done. One attempt to improve on equation (4.27) is described briefly below.

The suboptimal fit of equation (4.34) to urban catchment data might be due to the assumption of a fixed 70% runoff from impervious surfaces, or the assumption of the rural percentage runoff from pervious surfaces within the urban area. In order to test this, the percentage runoff data used by Kidd and Lowing (1979) in their analysis of small (fully sewered) urban catchments was used to derive a new percentage runoff equation for urban areas in terms of SOIL, I and CWI (not the UCWI term used by Kidd and Lowing).

The resulting equation is given below

$$PR_u = 21\ SOIL + .11\ (CWI - 125) + .73\ I \qquad (4.28)$$

where I is percentage impervious.

This equation was then combined with equation (4.25) for the rural percentage runoff giving:

$$PR = (1\text{-}URBAN)\ PR_r + (URBAN)\ PR_u \qquad (4.29)$$

Because of its form, equation (4.29), yields a good fit to rural catchments at one end of its range and to fully urban catchments at the other. Applied to the FSR data set, the optimum relationship between I and URBAN was found to be $\approx$ 30 URBAN, but the resulting fit was not quite as good as equation (4.27). Moreover, substituting typical values of SOIL and CWI showed that in many cases equation (4.29) predicted lower percentage runoff in urban catchments. This somewhat surprising result may be explained by the different character of events used in deriving the percentage runoff equations for rural and urban catchments. The rural equation (4.25) was derived on fairly long events, often with high values of CWI; the urban equation (4.28) was derived from short predominantly summer thunderstorm events corresponding to low values of CWI. From this analysis it was concluded equation (4.29) gave no improvement over equation (4.27).

To summarise, it is recommended that equation (4.27) is used to estimate the effect of urbanisation on percentage runoff. This equation gives a better fit to rural catchments, but the fit to urban catchments is not much better than the original FSR equation (4.23). Equation (4.27), 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 or perhaps equation (4.29). Equation (4.27) 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 Institute of Hydrology (1978).

4.3.3 Average Non-Separated Flow

The FSR equation for Average Non-Separated Flow (equation 4.24) did not include the variable URBAN. Subsequently an equation has been derived including URBAN

TABLE 4.6

Catchment No.	T_p	URBAN	RSMD	Stream Slope (m/km)	Stream Length (km)
RURAL CATCHMENTS					
20001	8.06	0.0	35.92	6.03	31.90
21005	8.06	0.0	43.96	4.68	35.88
23002	5.74	0.0	42.34	10.67	22.80
23005	6.51	0.0	44.59	4.94	36.70
24003	4.44	0.0	50.56	14.33	21.02
24007	4.66	0.0	36.61	14.89	11.90
25003	3.25	0.0	56.42	35.79	5.07
25810	1.90	0.0	58.15	64.40	.27
27001	8.83	0.0	34.52	4.01	65.09
27010	4.65	0.0	33.67	30.64	9.25
27027	4.76	0.0	51.40	4.46	55.05
27034	10.45	0.0	53.85	4.10	50.25
28016	21.05	0.0	26.71	2.47	38.60
28023	8.84	0.0	45.24	10.04	26.41
28033	2.31	0.0	46.55	33.37	3.90
28801	3.02	0.0	45.50	32.61	4.18
29001	7.90	0.0	26.08	3.33	20.17
30001	19.72	0.0	25.63	1.88	47.90
31006	18.45	0.0	22.18	3.04	37.07
32801	4.48	0.0	23.97	14.26	2.41
33014	25.42	0.0	20.35	2.23	29.19
33015	18.97	0.0	22.12	1.03	39.08
33024	21.50	0.0	21.33	2.24	32.50
33029	7.91	0.0	22.97	1.63	7.04
33045	17.45	0.0	19.66	2.51	8.54
33809	17.01	0.0	17.71	1.65	19.00
34003	13.51	0.0	22.37	2.31	20.06
34005	28.98	0.0	22.91	1.74	22.65
34007	16.83	0.0	20.77	1.47	16.57
34011	17.90	0.0	22.50	1.24	13.40
35008	10.43	0.0	20.86	2.88	11.93
36008	22.14	0.0	20.51	1.71	37.94
37003	17.64	0.0	18.65	2.40	25.03
37008	25.17	0.0	21.03	1.36	43.53
38003	3.92	0.0	22.46	2.59	17.33
39017	10.28	0.0	23.61	4.82	7.08
39022	21.97	0.0	29.72	.65	51.88
39025	11.91	0.0	28.65	3.20	23.19
39026	20.20	0.0	22.14	2.11	27.88
39813	7.21	0.0	34.63	7.43	4.06
40006	6.65	0.0	30.32	6.20	13.51
40007	12.57	0.0	31.89	2.20	26.31
40008	16.37	0.0	27.38	1.90	29.83
40009	8.38	0.0	28.52	3.20	19.89
40010	16.57	0.0	28.58	2.60	27.68
41005	17.10	0.0	30.34	3.50	19.55
41006	12.92	0.0	31.04	3.20	14.24
41007	27.36	0.0	29.41	1.20	44.39
45002	7.18	0.0	50.49	5.70	48.06
45003	11.42	0.0	41.49	5.93	27.43
45804	4.80	0.0	57.53	6.90	36.78
45805	6.43	0.0	56.93	8.30	33.76
46003	4.58	0.0	71.86	6.50	35.15
46005	3.58	0.0	81.61	22.60	11.82
46802	2.26	0.0	74.40	26.90	3.62
46901	2.03	0.0	84.23	75.90	2.83
47007	5.29	0.0	57.61	17.80	16.61
49003	4.48	0.0	55.32	10.80	6.67
52004	6.89	0.0	40.82	5.10	14.25
52005	9.71	0.0	34.76	5.60	37.31
52006	9.84	0.0	38.65	5.50	16.72
52010	9.39	0.0	31.81	2.80	30.44
52805	3.35	0.0	32.60	12.20	6.33
53005	9.50	0.0	36.58	3.00	24.58
53007	10.02	0.0	32.77	2.30	27.71
53008	12.74	0.0	28.48	1.60	25.07
53009	7.97	0.0	37.10	4.00	21.77

Catchment No.	T_p	URBAN	RSMD	Stream Slope (m/km)	Stream length
54007	13.96	0.0	21.98	3.30	36.17
54010	16.67	0.0	22.84	2.90	38.95
54011	12.65	0.0	22.93	4.87	27.00
54019	38.25	0.0	22.02	1.40	56.66
54020	17.06	0.0	26.85	2.45	31.85
54022	1.90	0.0	81.79	67.00	4.58
55008	2.03	0.0	82.42	36.30	7.32
56003	4.00	0.0	52.47	9.02	20.21
56006	4.45	0.0	62.90	7.87	22.46
57004	6.69	0.0	79.47	7.30	25.75
57005	6.52	0.0	74.88	9.23	42.26
58011	4.32	0.0	56.57	10.33	20.15
58002	3.97	0.0	80.19	13.50	28.33
58003	5.77	0.0	42.60	9.25	13.55
60002	6.70	0.0	58.02	4.82	47.04
60007	3.90	0.0	76.86	10.15	34.15
61001	7.09	0.0	45.49	2.74	26.43
64001	5.62	0.0	64.03	5.22	37.47
65001	5.55	0.0	109.14	33.37	15.28
65801	3.57	0.0	110.58	52.71	4.49
66001	4.18	0.0	66.13	17.07	28.84
66002	6.30	0.0	43.41	6.03	45.58
67005	5.25	0.0	44.06	10.72	29.15
67008	6.23	0.0	32.12	5.31	43.18
68006	4.00	0.0	36.01	9.41	31.48
68802	4.57	0.0	29.65	3.43	4.81
69008	7.26	0.0	38.63	9.95	22.75
71003	4.02	0.0	64.72	35.60	5.15
71802	7.37	0.0	54.21	4.80	43.11
71804	1.82	0.0	64.21	28.52	7.52
72002	5.60	0.0	44.53	3.69	40.82
73804	6.85	0.0	98.10	13.63	10.95
74001	3.42	0.0	88.53	13.82	19.58
76008	6.20	0.0	37.92	5.82	55.27
76011	2.31	0.0	45.77	25.49	1.90
77011	5.68	0.0	47.70	3.72	55.53
84002	2.97	0.0	72.59	28.50	6.42
85002	4.62	0.0	55.88	8.73	34.86
99001	21.69	0.0	32.26	2.96	40.00
URBAN CATCHMENTS					
19002	7.40	10.0	41.32	5.06	17.89
19005	6.10	10.0	39.88	5.49	30.24
24005	7.07	10.0	33.79	5.36	30.27
25004	11.41	10.0	28.39	1.82	44.21
27026	8.05	10.0	32.36	3.83	15.83
28026	26.78	10.0	25.81	1.36	34.07
37001	30.56	10.0	15.87	1.22	62.60
37007	13.37	10.0	19.05	1.70	24.63
37807	6.28	20.0	15.59	2.50	12.41
38007	3.09	10.0	19.79	7.47	5.60
39004	1.70	60.0	27.07	4.36	2.39
39005	3.07	80.0	23.21	2.28	7.40
39814	1.63	90.0	35.09	3.96	3.46
39820	5.85	50.0	23.38	4.48	10.75
39830	2.86	60.0	24.75	10.11	5.26
39831	.87	40.0	23.81	16.12	4.01
41801	3.48	40.0	31.58	19.60	3.51
41811	7.93	10.0	30.46	4.80	10.21
54004	11.46	20.0	22.81	1.92	28.81
54006	25.90	10.0	25.05	.80	72.56
56005	4.12	10.0	62.90	13.36	26.82
69011	4.90	30.0	36.89	9.07	17.54
84008	4.00	30.0	43.94	13.45	18.89
84012	7.30	30.0	42.20	6.19	31.13

$$ANSF = .001 \{ .376(CWI-125) + .75 \; RSMD + 15.0 \; URBAN + .50 \} \quad (4.30)$$

The correlation coefficient of equation (4.30) compared with equation (4.24) has increased from 0.668 to only 0.673. However, the coefficient of URBAN is significant at the 0.1% level and the change in constant and CWI coefficient are both more than 2 standard errors. It may therefore be stated with some confidence that ANSF is higher in urban catchments than their rural counterparts. This conclusion may seem at odds with intuition, since it is generally considered urbanisation reduces baseflow, but it must be borne in mind that ANSF is not just "baseflow" recession, but includes some "slow" response runoff having concentration time greater than 4 x LAG as defined in the FSR. Urbanisation may shorten the overall lag, but rural areas within the catchment may continue to respond at the same rate. Thus some response that, in a rural catchment, would be considered relatively fast (concentration time less then 4 x LAG) may, in an urbanised catchment, be considered slow (concentration times more than 4 x new LAG) and included in ANSF.

Further analysis of ANSF using separate urban and rural subsets has suggested an even more significant increase in ANSF with URBAN (coefficient 30.0) but in view of the generally small level of ANSF compared with the hydrograph peak, and in view of the small improvement in explained variance, the original FSR equation (4.24) is recommended in preference to equation (4.30).

4.3.4 Unit hydrograph time to peak

The FSR equation for unit hydrograph time to peak (equation 4.18) includes the factor (1 + URBAN) raised to the power -1.99, implying that fully urbanising a catchment reduces time to peak by 75%. This figure is similar to results obtained by other workers, but since again regression coefficients should not be used to estimate the effect of changes in the catchment characteristics, an analysis similar to that applied previously to mean annual flood, percentage runoff and ANSF was performed - considering separate urban and rural subsets.

The original data set of 130 catchments was separated into 106 rural catchments and 24 urban catchments (URBAN $\geq$ .1) - see table 4.6 - and the following equations derived.

$$Tp_r = 59.5 \; S1085^{-.38} \; RSMD^{-.45} \; MSL^{.10} \quad (4.31)$$

$$Tp_u = 9.7 \; S1085^{-.36} \; RSMD^{-.25} \; MSL^{.45} \; (1 + URBAN)^{-1.12} \quad (4.32)$$

Full details of the regressions are given in Table 4.7.

In equation (4.32), only the exponents of slope and stream length are significant at the 5% level, and moreover only the exponent of length is more than one standard error different from the original FSR equation (4.18). The lower value for the exponent of URBAN is somewhat compensated for by a lower value for the constant, and may be due to the lack of rural catchments in the data set to give an URBAN = 0 origin. In equation (4.31, the exponent of slope is just not significant at the 5% level, but

TABLE 4.7 Time to peak regression details for urban and rural subsets

RURAL CATCHMENTS: NO OF OBSERVATIONS - 106

Variable Name	Coeff	Standard Error	t Statis	R^2	Standard Error of Est.	Constant	Antilog of Const.
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	-	-	-	-

URBAN CATCHMENTS: NO OF OBSERVATIONS - 24

Variable Name	Coeff	Standard Error	t Statis.	R^2	Standard Error of Est.	Constant	Antilog of Const.
LOG(S1085)	- 0.36	0.15	2.41	0.92	0.15	0.98	9.7
LOG(RSMD)	- 0.25	0.30	.81	-	-	-	-
LOG(MSL)	0.45	0.15	2.96	-	-	-	-
LOG(1 + URBAN)	- 1.12	0.64	1.75	-	-	-	-

all exponents are within one standard error of the original FSR equation, suggesting any interaction between URBAN and the other catchment characteristics is within the general noise level. Furthermore, substituting the rural equation (4.31) into the original FSR equation gives

$$Tp = Tp_r\{0.783\ RSMD^{.03}\ MSL^{.04}\ (1 + URBAN)^{-1.99}\} \qquad (4.33)$$

which substituting typical RSMD and L values for urban catchments (30, 20) gives:

$$Tp = .98\ Tp_r (1 + URBAN)^{-1.99} \qquad (4.34)$$

The original FSR equation is thus considered able to represent the effects of urbanisation satisfactorily. Its use is preferred to equations (4.31) and (4.32) because it avoids the dichotomy of separate equations for urban and rural catchments, and because the exponents in the urban-only equation are subject to large uncertainties.

To test further whether equation (4.18) adequately accounts for urbanisation, the urban data set was "ruralised", multiplying by $(1+URBAN)^{1.99}$. An analysis of variance was then performed on the rural and ruralised data sets. Unhappily, the two sets showed marginally significant (level 4.5%) differences in regression exponents. Ignoring these differences however, regression constants could be considered identical. The factor $(1+URBAN)^{-1.99}$ has therefore been partly substantiated, though improvements may be possible following further analyses.

It is interesting to note from equations (4.31) and (4.32) the increased importance of stream length and reduced importance of slope in urban catchments. This was also seen in Espey et al's (1969) equations (3.35) and (3.37). Moreover the length-slope combination in equation (4.32) tends towards the form $L/\sqrt{S}$ used by many workers in response time regressions (see appendix 3). The form $L/\sqrt{S}$ derives from the Manning and Chezy formulae where flow velocity is proportional to $\sqrt{S}$ and thus "concentration time" is proportional to $L/\sqrt{S}$. For comparison, regressions of Tp on $L/\sqrt{S}$ were also performed yielding the following equations.

$$Tp_r = 3.42\ (L/\sqrt{S})^{.39} \qquad (4.35)$$

$$Tp_u = 2.52\ (L/\sqrt{S})^{.56}\ (1 + URBAN)^{-.77} \qquad (4.36)$$

Compared to equations (4.31) and (4.32) the correlation coefficients have dropped to 0.72 and 0.90 respectively. Again the exponent of (1 + URBAN) was not significant. The exponents for $(L/\sqrt{S})$ are similar to those obtained by a number of workers (see Appendix 3). However, when RSMD is included as an independent variable, the exponents drop to 0.26 and 0.53, suggesting that interaction between RSMD and $L/\sqrt{S}$ is quite high.

Equations (4.35) and (4.36) have time to peak as the dependent variable. Most of the equations in Appendix 3 have the centroid-to-centroid lag time. In the FSR a different measure of the LAG was preferred, but the centroid-to-centroid time was found in deriving a Nash cascade unit hydrograph. For a better comparison with Appendix 3, catchment average values of this centroid-to-centroid lag time were found and regressed on $(L/\sqrt{S})$. The resulting equations had exponents of 0.23 and 0.36, smaller than those of equations (4.35) and (4.36), and smaller than all those in Appendix 3. Moreover, the inclusion of RSMD further reduces them to 0.1 and 0.34. The results of this study suggest that $(L/\sqrt{S})$ is not sufficient to represent Tp or T_L alone, and the FSR equation (4.18) is to be preferred.

4.3.5 Unit hydrograph shape

Because urban areas within an urbanising catchment respond faster but rural areas continue to respond as before, one might expect urbanisation to yield 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.

$$(QpTp)_r = 162 + 2.6\ Tp \qquad (W/Tp)_r = 1.39 - .0083\ Tp \qquad (4.37)$$

$$(QpTp)_u = 165 + 2.5\ Tp \qquad (W/Tp)_u = 1.39 - .0079\ Tp \qquad (4.38)$$

Comparing these equations with the original FSR equations (4.19) and (4.20) showed corresponding coefficients were all within one standard error of each other. Consequently the effect of urbanisation on unit hydrograph shape is considered negligible, and the final FSR relationships equations (4.21) and (4.22) are considered equally applicable to rural urban and urbanising catchments, i.e.:

$QpTp = 220$ $\qquad\qquad$ $TB = 2.525\ Tp$

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. Section 4.3.3 has already shown how ANSF is increased by urbanisation suggesting indeed that rural response is included in the "baseflow".

4.3.6 Design Conditions

The specification of design storm and antecedent condition in order to obtain the flood peak of required probability is a problem common to all isolated event rainfall-runoff models. In the FSR I Ch. 6.7 a simulation technique was used to obtain a set of specifications that on average 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, yielding a steeper growth curve for flood peak than rainfall. This recommendation however was based on mainly rural catchments; urban catchments are generally less variable in response, and thus their flood frequency curves should tend more towards the corresponding rainfall frequency curves. Consequently a different choice of design conditions may be more applicable to urban catchments. Using data from 11 catchments the flood frequency curve implied by particular choices of antecedent condition and design storm were compared with both the observed flood frequency curve and, where available, the simulated flood frequency curve of FSR 1, Ch 6.7. Figures 4.6 show the fit for 8 catchments when the following choice of design conditions was made:

CWI - defined from SAAR as per FSR (see fig. 4.1)

Storm duration, D = $(1 + SAAR/1000)T_p$ as per FSR
Storm depth, P = the depth in duration D having return period equal to that of required flood
Storm profile - 50% summer (see FSR Vol II Ch. 6 and Fig. 4.7)

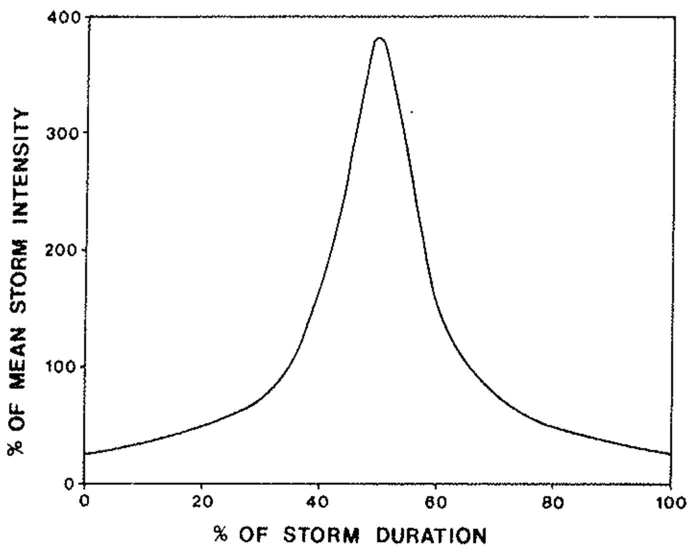
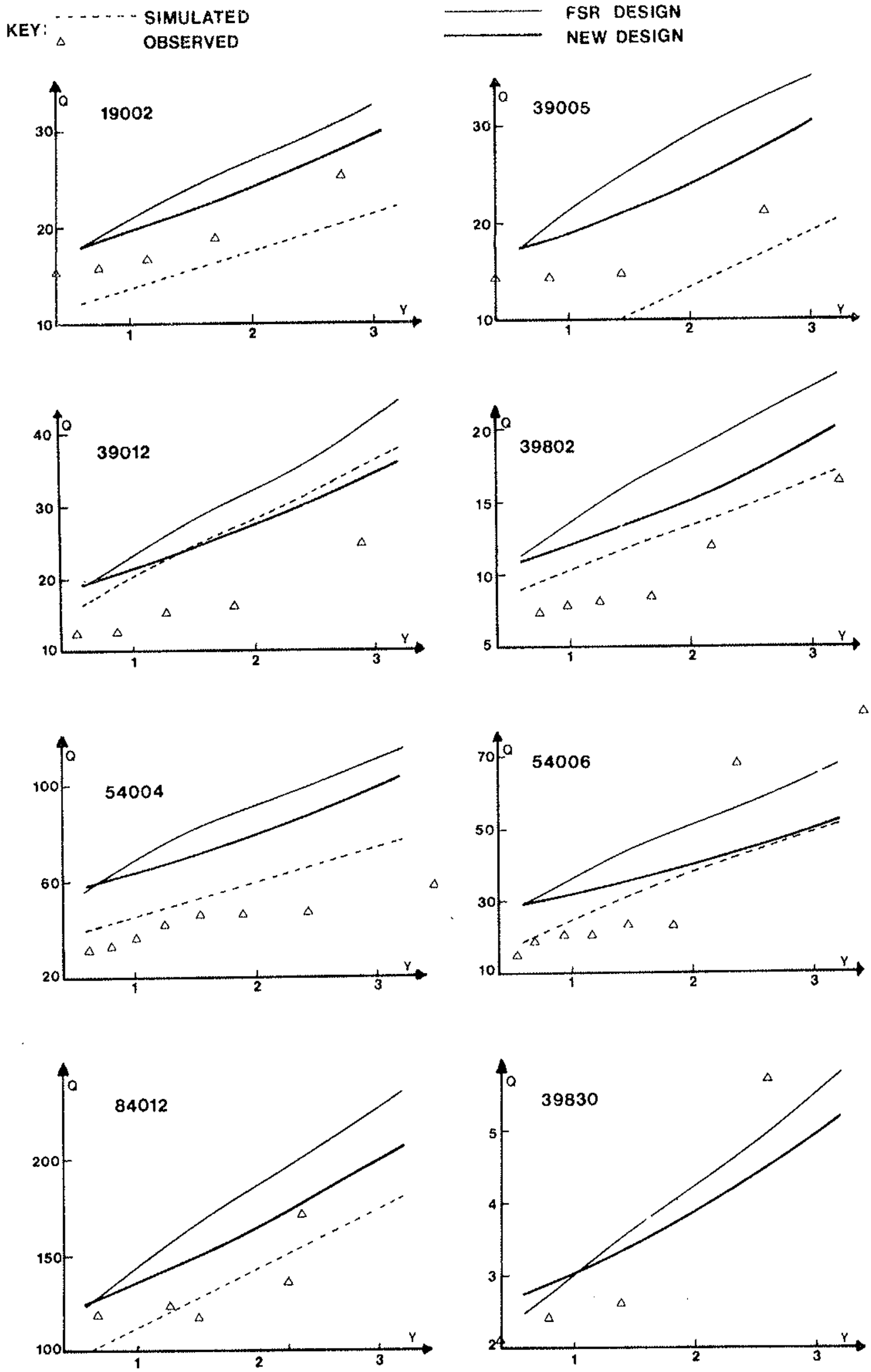


FIGURE 4.7

50% summer rainfall profile

FIGURE 4.6 Comparison of observed, simulated and design growth curves



Although this choice is seen to overestimate observed peak discharges on average, it does represent an improvement on the FSR design choices particularly with regard to the form of the growth curve. Perhaps a smaller value for CWI would improve the fit and be consistent with the observation of a shift in flood season to the summer months. However, in view of the small number of catchments available for this analysis it was considered unwise to depart too far from the FSR recommendations.

Compared with the FSR design conditions, use of equal return periods leads to a flatter flood frequency curve (up to the 500 year level at least) matching both intuition and such data as exist (see Section 4.2.3). The 50% summer rainfall profile is recommended in part for consistency with sewer design methods currently in use and under development in the UK. It leads to a slight increase in peak discharges, in most cases less than 5%.

Following the procedures of Institute of Hydrology (1979) the unit hydrograph method was reduced to a "Rational" formula

$$\hat{q} = RC.\frac{PR}{100}.\frac{P}{D}.\ AREA \qquad (4.39)$$

where $\hat{q}$ is the peak of the quick response hydrograph

and RC is a routing coefficient dependent on the ratio D/Tp.

Fig 4.8 is a plot of derived RC values against D/Tp when the 50% summer rainfall profile is input to the FSR triangular unit hydrograph. Also shown dotted is the same RC curve for the 75% Winter profile, showing the generally small difference in peak value obtained. Figure 4.9 is a plot of the range of complete design hydrograph shapes obtained using 50% summer rainfall profile and a range of D/Tp. Again, the curves for the 75% Winter profile are shown dotted.

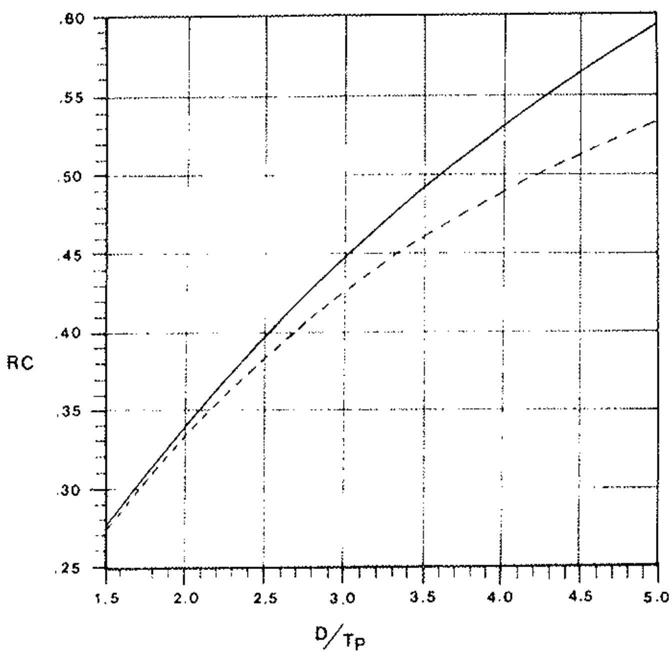


FIGURE 4.8

Routing constant for Rational formula

(solid line - 50% summer profile; dotted line - 75% winter profile)

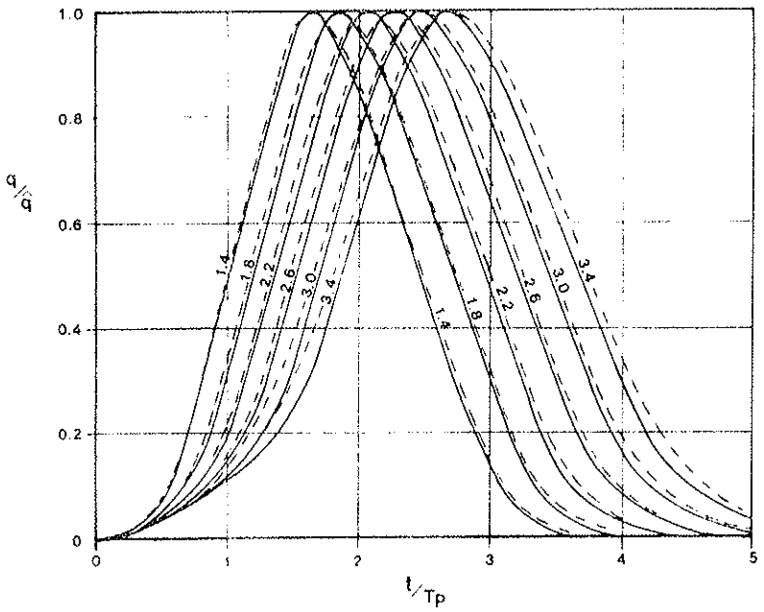


FIGURE 4.9 **Design hydrograph shapes for range of D/Tp** *(solid line - 50% summer profile; dotted line - 75% winter profile)*

The recommendation of a separate choice of design conditions for urban and rural catchments is contrary to the philosophy adopted in other parts of this report that there should be no break in techniques at some arbitrary degree of urbanisation. A dichotomy between urban and rural catchments has been introduced. It is recommended that the new design conditions should only be applied when the catchment is significantly urbanised and the major component of the flood peak derives from urban areas. Experience has shown this situation occurs when URBAN is greater than about 0.2. When the model is to be applied to pre- and post-urban conditions, equal return periods should only be applied to the post-urban case, however, for convenience, the 50% summer rainfall profile may be applied to both pre- and post-urban cases without any appreciable difference in predicting the effect of urbanisation.

4.4 A subcatchment approach to urbanising catchments

The unit hydrograph method discussed in Section 4.3 is a lumped approach to catchment modelling. However, as discussed in Section 1 of this report, urbanisation is a distributed phenomena, and its location within the catchment can have a significant effect on catchment response. To summarise, it affects firstly the relative scale of response from different parts of the catchment - urban development in the previously dry, non-contributing areas of the catchment will increase percentage runoff more than urban development in the previously wet, contributing areas. Secondly, it affects the phasing of response from different parts of the catchment - urban development upstream may cause the urban response to coincide with and reinforce the slower rural response from downstream, while urban development downstream may cause the urban response to pass before the rural response has arrived. In order to try to model these effects, some degree of distribution in catchment representation is required.

Following publication of the FSR, Price (1977) has combined the triangular unit hydrograph model with the Muskingum-Cunge channel routing of FSR III to develop a subcatchment model he calls FLOUT. The purpose of the model was principally to investigate river routing problems, and there was scope for improvement for application as a catchment simulation model. Consequently a new version of the model is under development at the Institute of Hydrology. However, a preliminary study of the potential advantages of a subcatchment model has been made using FLOUT, and a brief description follows.

4.4.1 FLOUT

FLOUT represents the catchment as a series of first and second order channel segments with discrete and distributed lateral inflows. Subcatchment routing uses the FSR unit hydrograph method and channel routing uses a variable parameter Muskingum-Cunge method. It is recommended that individual subcatchments should contain at least 10% of the total catchment area. The model allows different time periods for subcatchment and channel routing; the recommended time period for subcatchment routing is less-than-or-equal-to $T_p/5$ where T_p is the time to peak of the unit hydrograph, and for channel routing is L/2c where L is channel length and c is maximum wave speed. To fit the model to ungauged catchments, estimates of each subcatchment's time to peak, percentage runoff and baseflow are required. Also estimates of channel wave speed and attenuation are required. Wave speeds may be estimated from the difference in time to peak at upstream and downstream ends of the reach, and values for attenuation parameter may be estimated from channel breadth and slope. Fuller details of the model and fitting methods can be found in Price (1977).

4.4.2 Comparison of FLOUT with FSR lumped unit hydrograph model

FLOUT was fitted to the 30.3 km^2 Silk Stream catchment in North London. Rainfall-runoff data were available for the period 1928 to 1944, during which time the extent of urban area in the catchment grew from 0.4 to 0.56. The new urban area was however located towards the outfall of the catchment as shown in Fig. 4.10.

The catchment was divided into 7 separate subcatchments and subcatchment unit hydrographs derived using the FSR equations. Travel times through river reaches were estimated by applying the FSR time to peak equation (4.18) to the upstream and downstream end of the reach, assuming rural conditions (URBAN = 0). The estimate of travel time was then adjusted for urbanisation using the factor (1 + URBAN) to the exponent - 1.99.

Eight events were chosen from the record for analysis, each event being of a significant size-in 5 cases the annual maximum. Of these 8 events, four were chosen to represent the early urbanisation record, and four the later urbanisation record. In each group of four events, two

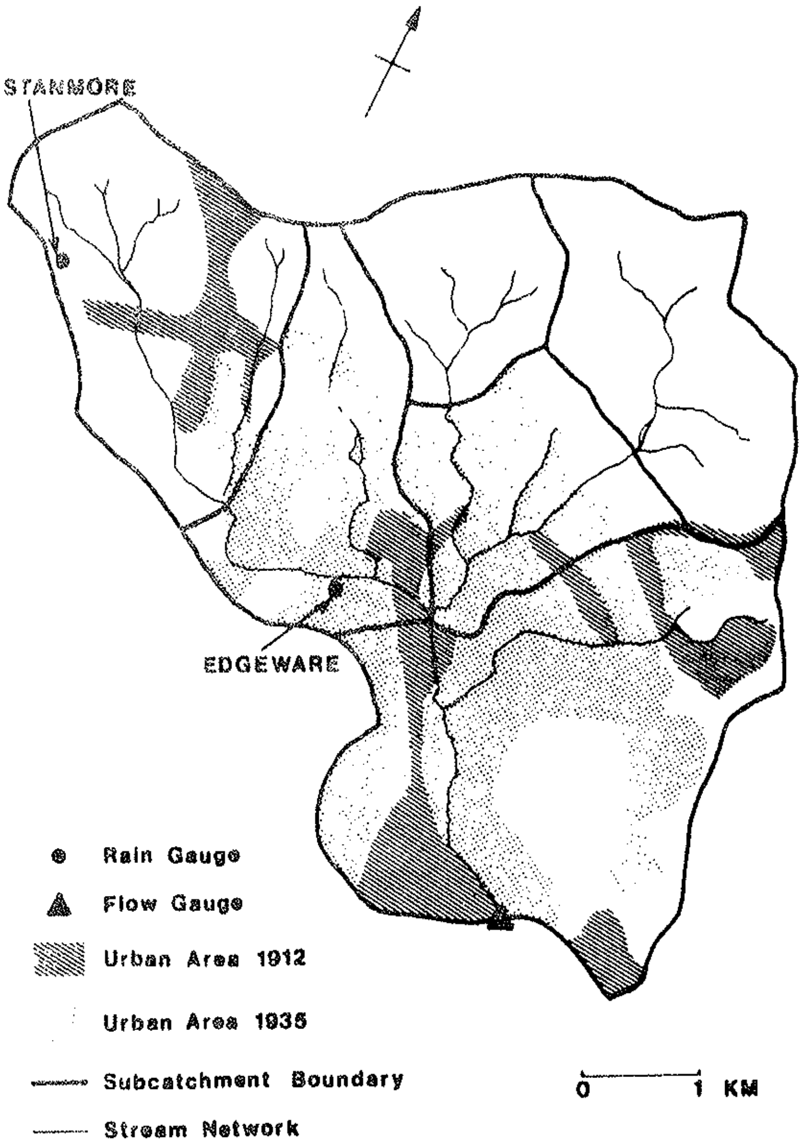
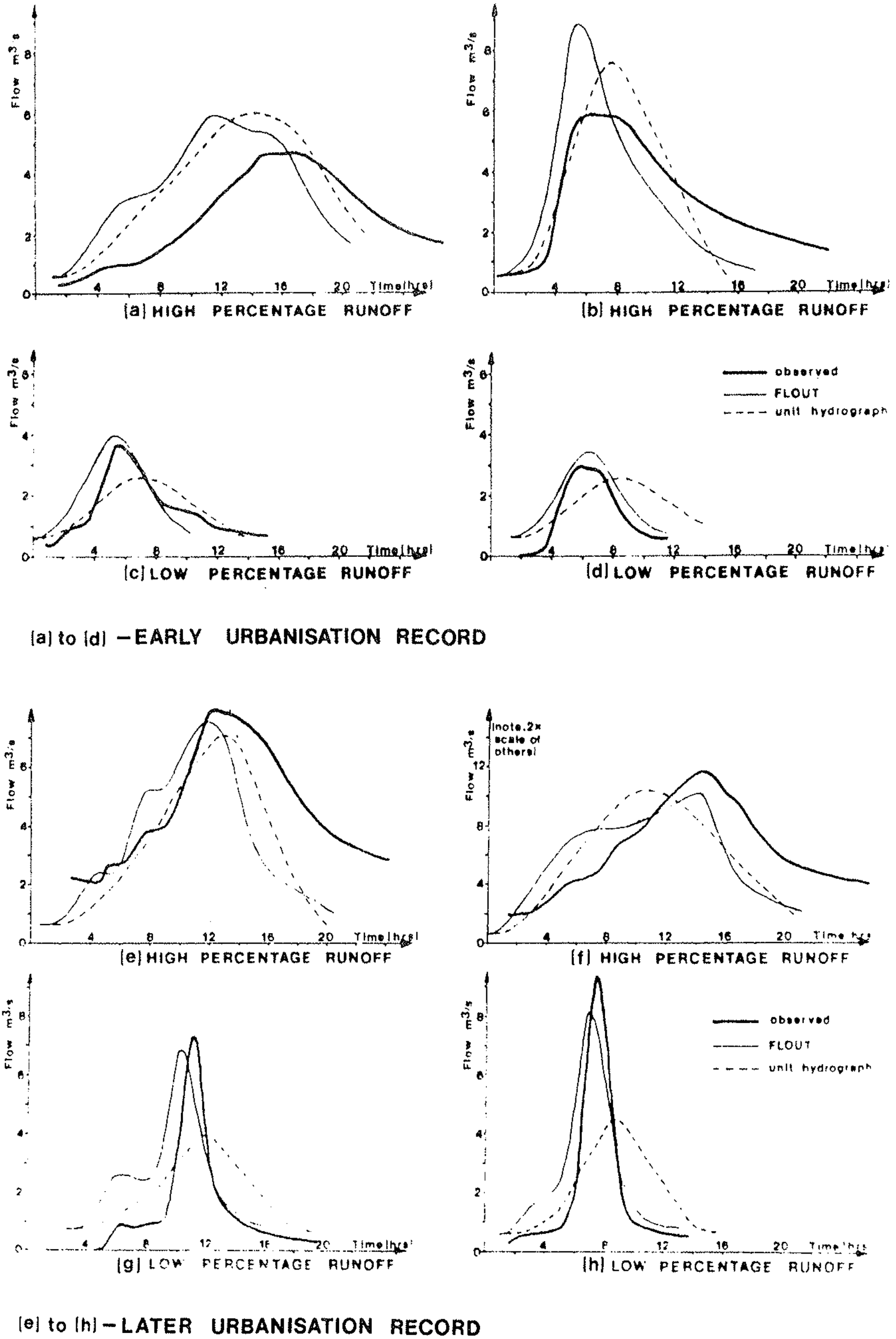


FIGURE 4.10 Silk Stream catchment

represented high percentage runoff events and two lower percentage runoff events. Low percentage runoff does not imply low peak flow; 3 of the low percentage runoff events were still the annual maximum events. Percentage runoff for each modelled event was set equal to the observed percentage runoff, and was distributed between the subcatchments according to the following rule: 80% runoff (an earlier recommendation) was assumed from paved area, and the balance was distributed equally to pervious areas.

Figure 4.11 a-h shows the observed hydrographs and predicted hydrographs using FLOUT and the lumped unit hydrograph procedure of the Flood Studies Report (N.E.R.C., 1975). It must be emphasized that neither model has been fitted to the event except in terms of direct runoff volume; otherwise the models have been fitted using only catchment characteristics and published regression equations. It is expected that base flow estimates would be improved if values for Antecedent Catchment Wetness

FIGURE 4.11 Comparison of FLOUT and lumped unit hydrograph models



Index were available. Apart from perhaps events 2 and 3 the FLOUT simulation represents an improvement over the lumped unit hydrograph simulation, particularly for the low percentage runoff events (nos 5, 6, 14, 17) when little runoff from the upstream subcatchments is predicted. FLOUT follows the form of the observed response very well, but the tendency to overpredict the beginning of the hydrograph and underpredict the recession suggests some improvement in effective rainfall separation could be made. Overall it is considered FLOUT is a better representation of catchment response, which was sufficient justification for further development. It is hoped such development will be reported in the future.

5. CONCLUSIONS AND SUMMARY OF RECOMMENDATIONS

The first part of this report has discussed the expected effect of urbanisation on flood magnitude and frequency: an increase in volume of runoff and a reduction in flow travel times; a flood hydrograph that is faster to peak, faster to recede, and of increased peak discharge; and an increase in the frequency of floods exceeding specified levels. Four factors were identified as of possible significance to the exact magnitude of the changes expected: the typical rural response before urbanisation; the severity (frequency) of the storm event; the typical rainfall regime; and the location of urban area within the catchment. Consideration of these factors was suggested by an extensive review of literature given in Section 3 of this report. This review considered relatively simple methods of estimating the effects of urbanisation (regression equations for 2, 5, 10 (etc) year floods, simple rainfall runoff models) and more complicated simulation models. Simple models were necessary to give a feel for the size of the problem. Also, since simple models were more easily applied, there was considerable experience in their use, and results typical of many regions had been published. Simple models were also more "analytical" in that an open mind on the effects was kept until a later stage. Simulation models however were based on the premise of certain expected changes and little basic analysis was done, but more the determination of optimum parameters. However, the potential of simulation models to account for the four factors specifically mentioned above was much greater, and thus they seem well suited to estimating the effects of urbanisation.

The third and last part of this report has presented modifications to the widely used FSR methods (NERC, 1975) of flood estimation. The modifications permit more satisfactory estimation of the effects of urbanisation and may be summarised as:

<u>The mean annual flood approach</u>

(i) The effect of urbanisation on mean annual flood ($\bar{Q}$) may be estimated from

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

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

$\bar{Q}_r$ is the prediction from the FSR equation - (4.1 of this report)

URBAN is the fraction of the catchment under URBAN development

and PR_r is obtained from

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

where SOIL may be found from the revised SOIL map (Institute of Hydrology, 1978) and the relevant value for CWI is found from the FSR figure (4.1 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 increase 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 Section 4.2.2 of this report

The unit hydrograph approach

(i) The effect of urbanisation on unit hydrograph time to peak is adequately estimated by the existing FSR equation

$$Tp = 46.6\ \text{S1085}^{-.38}\ \text{RSMD}^{-.42}\ \text{MSL}^{.14}\ (1 + \text{URBAN})^{-1.99}$$

ie $$\frac{Tp_u}{Tp_r} = (1 + \text{URBAN})^{-1.99}$$

(ii) Urbanisation has no significant effect on unit hydrograph shape and the existing FSR triangular shape is recommended for both urban and rural catchments

$$QpTp = 220 \qquad TB = 2.525\ Tp$$

(iii) The effect of urbanisation on percentage runoff is oversimplistically accounted for in the FSR equation, and a better estimate may be derived from

$$PR_u = PR_r\ (1 - 0.3\ \text{URBAN}) + 21.0\ \text{URBAN}$$

or $$PR_u/PR_r = 1 + 0.3\ \text{URBAN}\ (\frac{70}{PR_r} - 1)$$

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

and P is total rainfall depth

(iv) Although urbanisation has an effect on the level of "Average Non Separated

Flow", the change is not significant in terms of flood peak and the FSR equation may be used for both urban and rural catchments.

$$ANSF = .001 \{.326 (CWI - 125) + .74 \, RSMD + 3.0\}$$

where RSMD is the 5-year return period effective rainfall of 1 day duration.

(v) To estimate the T-year flood in an urbanised catchment (URBAN > .2) the above unit hydrograph and percentage runoff equations should be combined with design input consisting of

CWI	- the same as given in the FSR (Fig. 4.1 of this report)
Rainfall Duration	- the same as given in the FSR (D = 1 + (SAAR/ 1000) Tp)
Rainfall Depth	- the depth in duration D having return period <u>equal</u> to that of the required flood.
Rainfall Profile	- the 50% summer profile (see Fig 4.7)

(vi) In addition to these recommendations, some consideration was given to a subcatchment unit hydrograph approach, but further development of the model was required before it was suitable for design use.

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JCP/JVP

APPENDIX 1 : SUMMARY OF INVESTIGATIVE RESULTS – from RIORDAN et al. (1978)

$XR\,(A,B)_T$ = the ratio of the storm runoff characteristic (X) for a watershed with A% of its area impervious and B% of its area sewered to the same storm runoff characteristic for the same watershed under rural conditions (approximately 0% impervious area and 0% sewered area) for the T year storm event (or for the watershed unit hydrograph when T = UH)

SYMBOL	STORM RUNOFF CHARACTERISTIC
X = Q	peak stormwater runoff rate
P	"time-to-peak" (time from the beginning of stormwater runoff to the peak stormwater runoff rate)
L	"lag time" (time from the centroid of excess rainfall to the centroid of direct stormwater runoff)
V	volume of direct stormwater runoff.

NOTES:

1. A blank (-) within the parentheses indicates that the investigator did not examine that parameter.
2. An asterisk (*) within the parentheses indicates that the storm runoff characteristic is relatively insensitive to the value of that parameter.
3. The investigators' qualitative description of the watersheds are listed when percentage values were not given and could not be estimated.

HP = high probability storm event
LP = low probability storm event
RES = residential development
SF = single-family development
PUD = planned-unit development
DEV = developed area
PD = partially-developed area
PS = partially sewered
var = various

YEAR	INVESTIGATOR	GEOGRAPHICAL AREA OF INVESTIGATION	AREA OF STUDY BASIN $(mi)^2$	EFFECTS OF URBANIZATION ON: Q_p	t_p & t_L	VOL
1955	BIGWOOD & THOMAS	Connecticut	4.1(min) 1545(max)	$QR(RES)_{2.33}=3.5\text{-}5.5$	--	--
1961	CARTER	Washington, D.C.	3.9(min) 546(max)	$QR(12,PS)_{2.33}=1.8$ $QR(12,100)_{2.33}=2.6$ $QR(100,100)_{2.33}=5.5$	$LR(*,PS)_{var}<.4$ $LR(*,100)_{var}<.2$	--
1961	WAANANEN	Northern NJ, MI, PA, & VA	varies	$QR(DEV,-)_{HP}=3\text{-}4$	--	--
1961	WIITALA	Detroit, MI	36.5 22.9	$QR(25,100)_{2.33}=2.3\text{-}2.7$	$LR(*,100)_{var}=.3$	$VR(25,100)_{var}\doteq 1$
1962	VAN SICKLE	Houston, TX	38(min) 204(max)	$QR(DEV,100)_{UH}=2\text{-}5$	$PR(DEV,100)_{UH}\doteq .1$	--
1963	SAWYER	Long Island, NY	31 10	$QR(DEV,-)_{2.33}>1$	--	--
1965	CRIPPEN	Palo Alto, CA (Sharon Creek)	0.4	$QR(PD,-)_{UH}=1.4$	$PR(PD,-)_{UH}\doteq 1$ $LR(PD,-)_{UH}\doteq .7$	--
1965	JAMES	Sacramento, CA (Morrison Creek)	72.7	$QR(30,30)_{2.33}=1.6$ $QR(30,30)_{100}=1.2$ $QR(100,100_{2.33}=4.5$ $QR(100,100)_{100}=3.1$	$PR(100,100)_{var}<1$	$VR(100,100)_{var}=$ 5.9-125
1965	ESPEY	Austin, TX (Waller Creek)	4.13	$QR(27,50)_{UH}=1.5$ $QR(50,100)_{UH}=2.1$	$PR(27,50)_{UH}\doteq .5$ $PR(50,100)_{UH}\doteq .4$	$VR(25,-)_{var}\doteq 2$
1967	WILSON	Jackson, MS	1(min) 10(max)	$QR(DEV,100)_{2.33}=4.5$ $QR(DEV,100)_{50}=3$	--	--

YEAR	INVESTIGATOR	GEOGRAPHICAL AREA OF INVESTIGATION	AREA OF STUDY BASIN $(mi)^2$	EFFECTS OF URBANIZATION ON: Q_p	t_p & t_L	VOL
1968	ANDERSON	Washington, D.C.	.0034(min) 570(max)	$QR(20,100)_{2.33}=3-4$ $QR(100,100)_{2.33}=6-7.7$ $QR(1-100,100)_{100}=2.4-3$	$LR(*,75)_{var}<.2$ $LR(*,100)_{var}\doteq.1$	--
1968	ESPEY	Houston, TX	varies	$QR(50,50)_{UH}=3$	$PR(50,50)_{UH}\doteq.3$	--
1968	MARTENS	Charlotte, NC & Central NC	.86(min) 865(max)	$QR(22,100)_{2.33}=2.6$ $QR(100,100)_{2.33}=4.7$ $QR(1-100)_{50}=1.9$	$LR(*,100)_{var}<.25$	--
1968	LEOPOLD	Compilation of Results BY: CARTER, WIITALA, JAMES, ESPEY, ANDERSON, WILSON, MARTENS.		$QR(20,20)_{2.33}=1.5$ $QR(20,100)_{2.33}=2.5$ $QR(100,100)_{2.33}\doteq6$	--	--
1969	LULL & SOPPER	Northeastern US, NH, MA, CT, NJ	4.5(min) 96.8(max)	$QR(DEV,-)_{var}>1$	--	--
1969	SARMA	Indiana	.05(min) 19.3(max)	$QR(40,-)_{UH}=1.7-1.9$	--	--
1969	KINOSITA	Tokyo, Japan (Syskuzii R.)	18.7	$QR(44,-)_{LP}=1.5$ $QR(100,-)_{LP}=2.5-4$	--	--
1969	RILEY	Austin, TX (Waller Creek)	4.13	$QR(40,-)_{UH}>1.3$	$PR(40,-)_{UH}<.8$	--
1969	SEABURN	Long Island, NY (East Meadow Brook)	31	$QR(28,65)_{UH}=2.5$	--	$VR(28,65)=1.1-4.6$
1970	FEDDES	Bryan, TX	1.39 1.98	$QR(24,-)_{UH}\doteq2$	$PR(25,-)_{UH}=.5$ $LR(25,-)_{var}=.6$	$VR(25,-)_{var}>1$
1970	STALL	E. Cen. IL (Boneyard Creek & Kaskaskia R.)	3.58 12.3	$QR(75,100)_{2}\doteq8$ $QR(75,100)_{50}\doteq4$	$PR(75,100)_{UH}\doteq.1$	--
1970	DA COSTA	--	--	$QR(90,-)_{var}3-12$	$PR(DEV,100)_{var}<1$	--
1971	REIMER	San Diego, CA (Los Coches Creek)	.06(min) 15(max)	$QR(DEV,50)_{100}=1.5-2.7$ $QR(DEV,100)_{100}\doteq2$	--	--
1972	RAO	Indiana & TX	.05(min) 19.3(max)	$QR(40,-)_{UH}=1.9$	$LR(40,-)_{var}\doteq.6$ $PR(40,-)_{UH}\doteq.4$	--
1973	JOHNSON	Houston, TX	.05(min) 358(max)	$QR(35,-)_{2}\doteq9$ $QR(35,-)_{50}\doteq5$	--	--
1976	STANKOWSKI	NJ	.6(min) 779(max)	$QR(80,-)_{2}=3$ $QR(80,-)_{100}=1.8$	--	--
1974	MCPHERSON	Schwippe Valley Germany	19.5	$QR(DEV,-)_{var}=2$	--	--
1974	DEMPSTER	Dallas, TX	--	$QR(40,-)_{2}=1.35$ $QR(40,-)_{50}=1.16$ $QR(100,-)_{50}=1.36$	--	--
1974	DURBIN	Santa Ana Valley, CA	3.7(min) 83.4(max)	$QR(DEV,-)_{2}=3-6$ $QR(DEV,-)_{100}\doteq1$	--	--
1975	BRAS	Puerto Rico (Hypothetical Catchment)	.02	$QR(50,100)_{10}=1.3-2$ $QR(50,100)_{50}=1.1-1.2$	$PR(50,100)_{10}\doteq.65$ $PR(50,100)_{50}\doteq.8$	
1975	DOEHRING	Southeastern New England	34(min) 219(max)	$QR(-,-)_{2.33}=1-1.6$ $QR(-,-)_{100}=1.2-2.3$	--	--
1975	MCCUEN	Baltimore, MD (Crayhaven Watershed)	.04	$QR(SF,-)_{2.25}\doteq2$ $QR(PUD,-)_{2-25}\doteq5$	--	--
1976	LAZARO	Unity, MD	72.8 34.2	$QR(DEV,-)>1$	--	--
1976	CECH	Texas Coastal Region (Houston, Galveston, Texas City)	varies	$QR(DEV,PS)_{2}=2-5$ $QR(DEV,PS)_{LP}=$ lesser effect	--	--

APPENDIX 2 : METRIC CONVERSION FACTORS

Length (L)

1 inch (in) = 25.4 mm
1 foot (ft) = 0.305 m
1 mile (mi) = 1.609 km

Area (A)

1 acre (acre) = 0.405 ha
1 sq mile (mi^2) = 2.590 km^2

Slope (S)

1 foot per mile (ft/mi) = 0.189 m/km

Length-Slope Ratio ($L/\sqrt{S}$, $LL_{ca}/\sqrt{S}$)

$1\ (mi/\sqrt{ft/mi}) = 3.701\ km/\sqrt{m/km}$
$= 117.0$ km

$1\ (mi^2/\sqrt{ft/mi}) = 5.955\ km^2/\sqrt{m/km}$
$= 118.3$ km

Discharge (Q)

1 cu foot per sec (ft^3/s) = 0.0283 m^3/s

APPENDIX 3: SOME SIMPLE LAG TIME RELATIONSHIPS

KEY:

T_L - centroid to centroid lag time (hr)

T_p - centroid to peak lag time (hr)

T_c - time of concentration (hr)

L - channel length (mi)

S - channel slope (ft/mi)

KIRPICH FORMULA:	T_c	$= 2.59\ (1/\sqrt{s})^{0.77}$	(overland flow)
CARTER FORMULA:	T_L	$= 3.1\ (L/\sqrt{s})^{0.6}$	(rural catchments)
		$= 1.2\ (L/\sqrt{s})^{0.6}$	(partially developed catchments)
		$= 0.54\ (L/\sqrt{s})^{0.6}$	(fully developed catchments)
WIITALA FORMULA:	T_L	$= 5.89\ (L/\sqrt{s})^{.60}$	(rural catchments)
		$= 1.90\ (L/\sqrt{s})^{.60}$	(urbanised catchments)
ANDERSON FORMULA:	T_L	$= 4.64\ (L/\sqrt{s})^{.42}$	(rural catchments)
		$= 0.90\ (L/\sqrt{s})^{.50}$	(partially developed catchments)
		$= 0.56\ (L/\sqrt{s})^{.52}$	(fully developed catchments)
MARTENS FORMULA:	T_L	$= 4.18\ (L/\sqrt{s})^{.52}$	(rural catchments)
		$= 1.83\ (L/\sqrt{s})^{.52}$	(partially developed catchments)
		$= 1.00\ (L/\sqrt{s})^{.52}$	(fully developed catchments)
PUTNAM FORMULA:	T_L	$= 6.76\ (L/\sqrt{s})^{.50}$	(1% impervious)
		$= 0.89\ (L/\sqrt{s})^{.50}$	(35% impervious)
WIBBEN FORMULA:	T_L	$= 2.25\ (L/\sqrt{s})^{.52}$	(all catchments)
KNIGHT FORMULA:	T_L	$= 5.19\ (L/\sqrt{s})^{.69}$	(rural catchments)
		$= 1.35\ (L/\sqrt{s})^{.69}$	(urban catchments)
HALL FORMULA:	T_L	$= 3.61\ (L/\sqrt{s})^{.42}$	(rural catchments)
		$= 1.10\ (L/\sqrt{s})^{.50}$	(ubranised catchments)
YOSHINO FORMULA:	T_L	$= 2.36\ (L/\sqrt{s})^{.7}$	(rural catchments)
		$= 0.34\ (L/\sqrt{s})^{.7}$	(urbanised catchments)

SNYDER FORMULA:	$T_p = 2.0\ (L\ L_{ca})^{0 \cdot 3}$	(rural catchments)
LINSLEY *et al* FORMULA:	$T_p = 1.20\ (L\ L_{ca}/\sqrt{s})^{0 \cdot 38}$	(mountain catchments)
(T_p from start of rainfall)	$= 0.35\ (L\ L_{ca}/\sqrt{s})^{0 \cdot 38}$	(valley catchments)
EAGLESON FORMULA:	$T_p = .067\ (L\ L_{ca}/\sqrt{s})^{0 \cdot 38}$	(urban catchments)
VAN SICKLE FORMULA:	$T_p = 0.9\ (L\ L_{ca}/\sqrt{s})^{0 \cdot 38}$	(rural catchments)
	$= 0.065\ (L\ L_{ca}/\sqrt{s})^{0 \cdot 38}$	(fully developed catchments)

APPENDIX 4 : SUMMARY OF CATCHMENT CHARACTERISTICS USED IN THE FLOOD STUDIES REPORT - from SUTCLIFFE (1978)

CATCHMENT CHARACTERISTICS

The assessment of floods at ungauged sites relies on the incorporation of data on certain catchment characteristics into the relevant formulae described earlier in the Guide. This appendix describes the characteristics used and their estimation.

The catchment characteristics and notations used are:

	Notations	Range of values at stations used in study		
		Minimum	Maximum	
Area	AREA	0 038	9868	km^2
Stream length	MSL	0.27	238.75	km (excluding Irish stations)
Stream slope	S1085	0.19	117.78	m/km
Stream frequency	STMFRQ	0.01	7.54	Junctions/km^2
Soil index	SOIL	0.15	0.50	
Lake index	LAKE	0.000	1.000	
Urban development	URBAN	0.000	0.808	
Annual average rainfall	SAAR	551	3454	mm
Net 1-day rainfall of 5-year recurrence	RSMD	15.6	117.5	mm

The range of values found at the stations used in the study indicates the likely magnitude of these catchment characteristics. Values near or beyond these limits should be checked for arithmetic errors.

It should be borne in mind when making estimates for ungauged catchments that regression equations are more precise in the middle of the range of data on which they are based. Unfortunately, the study has been able to draw on few records from small catchments, from the north of Scotland, from Northern Ireland, or from urban catchments. However, an important argument for a countrywide study is that it uses the widest possible range of records. The fact that the regression equation for the mean annual flood, for instance, is compatible with our physical knowledge of floods suggests that it is preferable to use the countrywide equations and if necessary adjust for a consistent bias in local records rather than rely on local analysis alone.

There are two instances where the lack of data suggest a separate approach for ungauged sites. Less than 5% of the records had an urban fraction over 0.25, and less than 10% had a lake index over 0.33. Estimates for ungauged sites with an urban fraction over about 0.25 should be based on urban drainage design methods, while estimates for catchments with a large proportion draining through a lake or reservoir should be adjusted by reservoir routing techniques.

AREA

The area draining to a site is the most fundamental catchment characteristic. It should be measured at an early stage in any study by planimetry or by counting squares from a map on which the watershed is drawn. Do not overlook any diversions or leats which increase or decrease the flood-producing area from the topographical catchment. The Ordnance Survey First Series 1:25 000 map should be used for deriving topographic characteristics as this was the series used during the investigation. Regressions will be revised or a conversion factor provided when the Second Series is complete.

Example: the River Almond at Craigie Hall (Grid reference NT 165752)

AREA = 369 km^2

STREAM LENGTH (MSL) AND STREAM SLOPE (S1085)

Main stream length (MSL or L) is derived during the assessment of stream slope (S1085). The stream slope is that defined by the United States Geological Survey as the mainstream slope between the 10 and 85 percentiles of mainstream length (upstream from the gauging station).

Raw data for both S1085 and MSL are derived as follows. Choose the main stream from maps which in most cases is simply the longest stream in the basin. In cases of difficulty, work upstream and at every junction follow the stream draining the larger area. Distances are measured upstream from the station with precision dividers set at 0.1 km (4 mm on the 1:25 000 map). It helps to mark every fifth step. Once the total length to the end of the stream is known, the lengths and elevations of the 10% and 85% points are used to calculate slope; the units used are parts per thousand or metres per kilometre. Stream length is sometimes described as that 'along the main channel between the gauge and the divide' which implies the length to the watershed. In the Flood Studies the channel was defined as the blue line on the 1:25 000 map. The variation in S1085 values is shown in Figure I.4.4 of the Report, p. I.299, which may help to prevent gross errors in calculation.

Example:

Number of steps (N) = 446; therefore MSL, the stream length = 44.6 km;

Mark the points 0.1 N and 0.85 N steps upstream from the starting point.

By interpolation between contours,
elevation at 0.85 N steps upstream = 625 ft
and at 0.1 N steps upstream = 90 ft
Difference = ΔH
= 535 ft = 163. 068 m

S1085 = ΔH/0.75 MSL
= 163.068/33.45 m/km
= 4.87 m/km

As a precaution against inaccurate setting of dividers, it is suggested that 50 steps be made along a straight line before and after map measurement and any departure from the expected 200 mm allowed for by a correction factor.

STREAM FREQUENCY (STMFRQ)

The channel network is described by 'stream frequency' simply measured by counting channel junctions on the 1: 25 000 First Series maps and dividing by basin area. The precise technique is as follows.

Once the necessary maps are assembled in a logical order, the stations or sites for estimation are marked to avoid duplication if more than one site is to be measured. The number of natural stream junctions is counted upstream from the lowest site, which is also included as a junction. It is best to work progressively up each tributary; the running total is noted at each major junction and at additional gauges. Artificial channels in fenland or flood plains and also canals are ignored. Where natural channels exist, but are not shown on the map, for instance in urban areas, or where junctions occur in a lake or reservoir, the missing junctions are counted.

In catchments under 0.2 km^2, the following procedure should be adopted to avoid exaggerated estimates. Move downstream to the nearest third order stream (see Figure I.4.5) and measure the stream frequency of its basin. Figure I.4.7 can be used as a rough check on the stream frequencies obtained by the user of the Report.

Example

$$\begin{aligned} N &= 375 \text{ junctions} \\ AREA &= 369\ km^2 \\ STMFRQ &= N/AREA \\ &= 1.02 \text{ junctions}/km^2 \end{aligned}$$

SOIL INDEX (SOIL)

The soil index is based on the soil map (Figure I.4.18) given in the Report, where five classes of soil are shown based on their 'winter rain acceptance potential'. Weights were ascribed to each soil class which indicate their individual runoff potential; a soil index for a catchment is derived by measuring the fractions of the catchment within each soil class, and adopting a weighted mean of these soil fractions (S_1, S_2, ...)

$$SOIL = \frac{0.15S_1 + 0.30S_2 + 0.40S_3 + 0.45S_4 + 0.50S_5}{S_1 + S_2 + S_3 + S_4 + S_5}$$

The areas of each class are determined by overlaying the soil map with a catchment map at the 1:625 000 scale. Sufficient accuracy is normally obtained by counting the squares on 1/10th inch graph paper.

Example

	Soil Class	No. of squares		
	1 (N_1)	0 x 0.15	=	0
	2 (N_2)	0 x 0.30	=	0
	3 (N_3)	0 x 0.40	=	0
	4 (N_4)	119 x 0.45	=	53.5
	5 (N_5)	28 x 0.50	=	14.0
Unclassified	(N_u)	0 Total (W)		67.5

$$N_s = N_1 + N_2 + N_3 + N_4 + N_5 = 147$$

$$\text{SOIL} = W/N_s$$

$$= \underline{0.459}$$

As a useful check, the soil index must lie in the range 0.15 to 0.50. A check on the catchment area is given by $(N_s + N_u) \times 2.52$.

LAKE INDEX (LAKE) AND URBAN INDEX (URBAN)

An index of lake storage and an index of urban development are used in some of the equations for predicting floods at ungauged sites. These indices are no substitute for lake or reservoir routing where the design site is immediately downstream of large storage, or for using urban runoff models where the flood runoff from a predominantly urban area is required. However, where the area draining through a lake or from urban development is not too high a proportion of the catchment area, results are improved by taking these into account rather than ignoring them.

All the lakes or reservoirs whose surface areas are less than 1% of the area contributing to that lake are ignored. In practice, each tributary is followed until a lake or reservoir is met whose area is greater than 1% of the area contributing; the contributing area is then recorded. It is unnecessary to continue upstream as a reservoir within this contributing area does not count. This is repeated on all other tributaries within the gauged catchment and all contributing areas are summed to give the total area contributing to lakes or reservoirs. This total contributing area is divided by the total area of the gauged catchment to give a lake index (LAKE). This index has the disadvantage, when used in a multiplicative equation, that the effect of increasing the lake fed fraction from 0.01 to 0.1 is the same as from 0.1 to 1.0. To overcome this and to provide an index which vanishes as a product when no lakes are present, the index is transformed from LAKE to (1 + LAKE) when used in regression analysis.

Examples

The surface area of two reservoirs in the Almond catchment exceed 1% of their respective catchment areas which total 14.8 km^2 or 0.040 of the total catchment area, therefore

$$\text{LAKE} = 0.04$$

Urban fraction is estimated from the area shown as built up on a suitable scale map, e.g. 1:63 360. This was estimated as 42.0 km^2 or 0.114 of the catchment, therefore

URBAN = 0.114

ANNUAL RAINFALL AND SHORT-TERM RAINFALL INDEX (SAAR & RSMD)

The standard annual average rainfall (SAAR), and the 1 day rainfall of 5 year return period minus the effective mean soil moisture deficit, (RSMD), are used as indices of catchment rainfall. The annual rainfall is an index of climate, while the net short-term rainfall is an index of flood-producing rainfall; in practice the two were found to be fairly closely related.

The annual average rainfall (SAAR) is obtained from Figure II.3.1. The average for the catchment may be obtained by sampling at about 20 points equally spaced on a grid overlay and taking the arithmetic mean. Alternatively, the weighted areas technique can be used.

SAAR = 914 mm

The calculation of RSMD requires M5-2 day rainfall from Figure II.3.2, the ratio r (M5-60 min/M5-2 day) from Figure II.3.5, and the effective mean SMD (SMDBAR) from Figure I.4.19. In all three cases, catchment average values are required and these may be obtained by grid point sampling or weighted areas.

The ratio M5-24 hours/M5-2 day is determined in terms of r, from Table II.3.7 (reproduced as Table I.6.21). M5-24 hours follows and this is converted to M5-1 day by dividing by 1.11 (Table II.3.1). The 1 day areal reduction factor (ARF) is obtained from Figure II.5.1 reproduced as Figure I.6.58 in the unit hydrograph example (Section 4). The 1 day ARF depends only on area as follows:

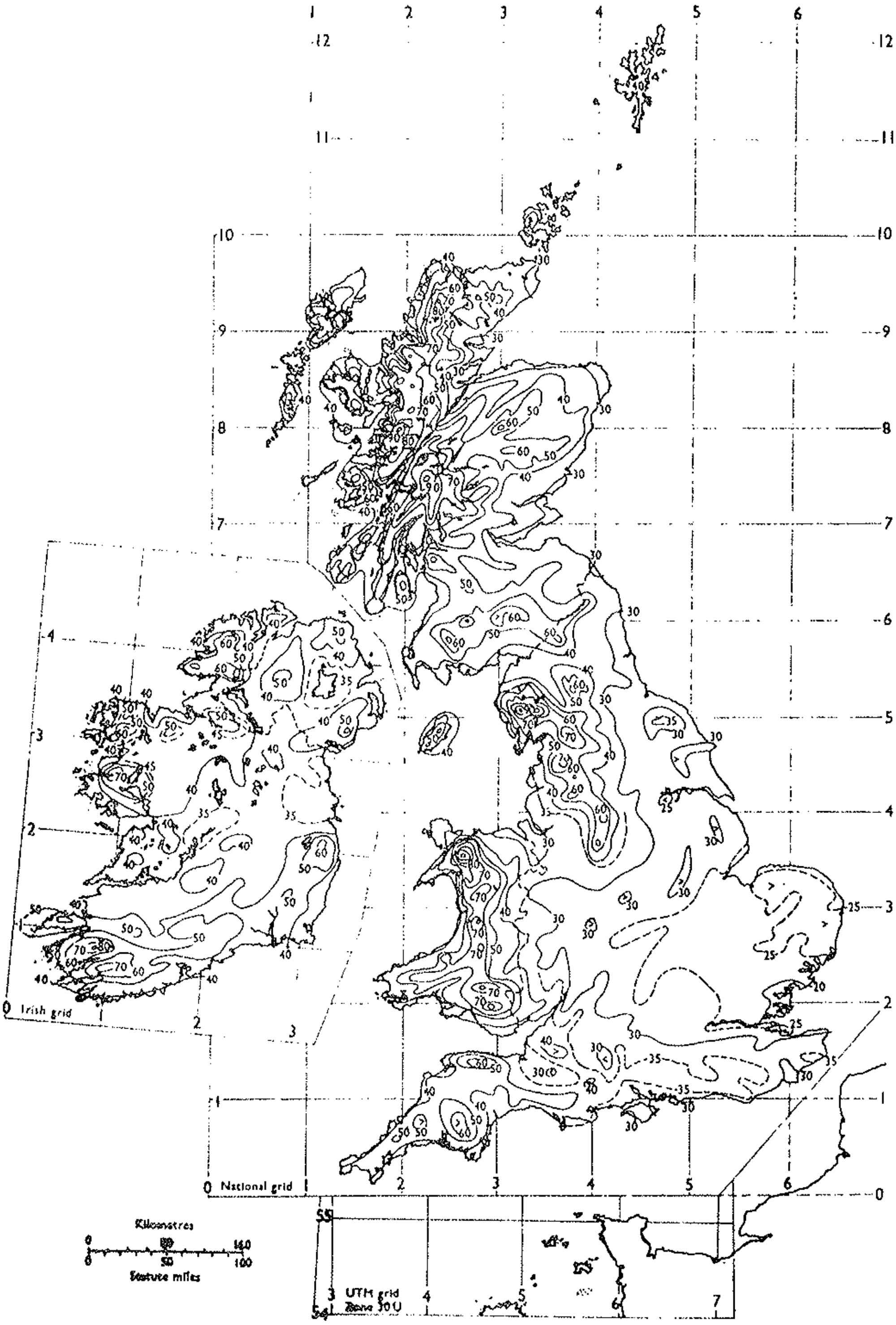
Area (km^2)	5	10	20	50	100	200	500	1000
ARF	0.980	0.975	0.970	0.955	0.940	0.925	0.910	0.880

Example

RSMD = M5-1 day x ARF - SMDBAR

r	=	25%
M5-24 hour/M5-2 day	=	82%
M5-2 day	=	57 mm
M5-24 hour	=	46.7 mm
M5-1 day	=	42.0 mm
ARF (1 day)	=	0.92
M5-1 day x ARF	=	38.6 mm
SMDBAR	=	6.6 mm
RSMD	=	32 mm

The relationship between RSMD and SAAR (standard annual average rainfall) as shown in Figure I.6.59 allows an initial estimate of RSMD to be made, as does the outline map of RSMD published in the ICE Flood Studies Conference 1975, p. 104, and reproduced overleaf.



Outline map of RSMD, the net 1 day rainfall of 5 year return period: values in millimetres, < denotes minimum area, > denotes maximum area

Appendix V: Derivation of $\bar{Q}_u/\bar{Q}_r$ equation from unit hydrograph method.

Institute of Hydrology (1979) have shown how the unit hydrograph method may be reduced to a simple "Rational Formula"

$$q = RC . \frac{PR}{100} . \frac{P}{D} . AREA \tag{1}$$

where q is the peak of the direct runoff hydrograph
RC is a routing coefficient dependent on the ratio D/Tp
PR is the percentage runoff
P is rainfall depth
D is rainfall duration
and Tp is the unit hydrograph time to peak

Considering equation (1) applicable to urban (suffix u) and rural (suffix r) conditions gives

$$\frac{q_u}{q_r} = \frac{RC_u}{RC_r} . \frac{PR_u}{PR_r} . \frac{P_u}{P_r} . \frac{D_r}{D_u} \tag{2}$$

Critical duration in equations (1) and (2) is taken as a fixed ratio of time to peak, depending only on average annual rainfall (see section 4.3.6)

$$D = (1 + SAAR/1000)Tp \tag{3}$$

Thus $(D/Tp)u = (D/Tp)_r$ and $RC_u = RC_r$. (4)

Also $D_r/D_u = Tp_r/Tp_u$. (5)

An expression for P_u/P_r may be found from the model of rainfall depth given in FSR Volume II p.26, which for durations longer than about half an hour may be written

$$P = KD^{1-n} \tag{6}$$

where n is an arbitrary exponent termed the rainfall continentality.

Substituting (4), (5) and (6) into equation (2); and neglecting areal reduction factor gives

$$\frac{q_u}{q_r} = \frac{PR_u}{PR_r} . \left(\frac{Tp_r}{Tp_u}\right)^n \tag{7}$$

Equation (7) gives the expected effect on hydrograph peak of a change in percentage runoff and a change in time to peak (together with a corresponding change in critical storm duration). Equations for PR_u/PR_r and Tp_r/Tp_u may be obtained from sections 4.3.2 and 4.3.4 as

$$PR_u/PR_r = 1 + \frac{I}{100} (\frac{PR_i}{PR_r} - 1) \qquad (8)$$

and $$Tp_r/Tp_u = (1 + URBAN)^2 \qquad (9)$$

Substituting these equations into equation (7) and assuming the ratio of direct runoff peaks is equivalent to the ratio of mean annual floods gives

$$\bar{Q}_u/\bar{Q}_r = (1 + URBAN)^{2n} \{1 + \frac{I}{100} (\frac{PR_i}{PR_r} - 1)\} \qquad (10)$$

FSR Volume II p.26 gives n values in terms of average annual rainfall. Since most areas in the UK suitable for urban development have average annual rainfall of between 500 and 1000 mm, an average value of $n = 0.75$ may be taken. Section 4.3.2 recommends taking $I = 30.URBAN$ and $PR_i = 70\%$. PR_r may be estimated from a simplified form of equation 4.

$$PR_r = 102.4\ SOIL + 0.28\ (CWI - 125) \qquad (11)$$

Equation (10) then reduces to

$$\bar{Q}_u/\bar{Q}_r = (1 + URBAN)^{1.5} \{1 + 0.3\ URBAN\ (\frac{70}{PR_r} - 1)\} \qquad (12)$$