Institute of Hydrology

Selection of design storm and antecedent condition for urban drainage design

by

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ABSTRACT

A fundamental problem in design flood estimation using isolated event simulation models is the selection of a suitable combination of design storm and antededent conditions. Current practice in urban storm drainage design is to adopt arbitrarily a storm duration, profile and catchment wetness, and to assume that the return periods of rainfall depth and flood peak are equal. This report describes the analysis that was used with the newly developed Wallingford model to examine the relationship between rainfall and flood return periods, and thus to determine systematically a suitable set of design inputs which give a peak runoff of the required return period. Using data from 2 real catchments and 3 imaginary catchments, synthetic flood frequency distributions were derived by simulation. A subsequent sensitivity analysis of the model to variations in design inputs allowed definitions to be found for the rainfall duration and profile and the catchment wetness that ensured equal probabilities of rainfall depth and runoff peak. This work permits confidence in the use of the Wallingford model for storm drainage design in the UK.

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CONTENTS

1. INTRODUCTION

Background

When rainfall-runoff models are used in drainage design to estimate the flood discharge due to some design storm, it is usually implicitly assumed that the flood discharge has the same probability of exceedance as the design storm. This applies to both natural catchment and urban catchment models, and is true of the two methods of storm-sewer design currently recommended and in use in the UK (TRRL, 1976; Colyer, 1975) - the Rational Formula and the TRRL Method (Watkins, 1962). It is being increasingly recognised that the proper selection of design storm and antecedent wetness conditions is of paramount importance if the probabilities of rainfall and runoff are indeed to be considered equal. The relationship between the probabilities of the design output (flood discharge) and design inputs has been investigated and is described in this report.

To illustrate the problem, consider the effect of catchment wetness. The peak discharge caused by rainfall on a dry catchment will be smaller than that caused by the same rainfall on a wet catchment. Put another way, a 10-year flood may be the result of rainfall anywhere between, say, one year and 40-year return period, given the infinite number of possible combinations of the other design inputs. There is, therefore, no unique relationship between the return periods of rainfall and runoff, and the study described in this report sets out to find a stable set of the design inputs for which, on average, the postulate "rainfall return period = runoff return period" holds true.

The Hydraulics Research Station and the Institute of Hydrology are collaborating in the development of improved methods for the hydraulic design of storm sewers (The Wallingford Model). These methods will be included in the Manual of Good Working Practice currently in preparation by the National Water Council/Department of the Environment Working Party on the Hydraulic Design of Storm Sewers. Four hydrological variables are input to the Wallingford Model: rainfall volume, rainfall duration, the temporal distribution of rainfall (a storm profile), and an index of antecedent catchment wetness (UCWI) formed from a combination of soil moisture deficit and 5-day antecedent precipitation index. The objective of this investigation is to obtain a stable set of these inputs which will, on average, produce the flood peak of required exceedance probability. An attempt is also made to adjust this set according to the climatic regime of any specified location in the UK. The model used in this study is described in more detail in section 2.

Previous work

To estimate a design discharge of specified exceedance probability, some workers have abandoned the use of a design storm, and have turned to continuous simulation (Crawford and Linsley, 1966; Leclerc and Schaake, 1973; Marsalek, 1977). A long rainfall record, either observed or synthetic, is fed through the rainfall-runoff model to generate a complete synthetic flow record. Subsequent flood frequency analysis of this record allows estimation of discharges for a range of frequencies. However, the procedure is lengthy and costly, and may not be suitable for design since the flood frequency distribution obtained depends to some extent on the catchment configuration upstream. Several complete simulation runs with different trial designs may be necessary.

Another approach was adopted in the Flood Studies Report (NERC, 1975). Here, sensitivity analysis is used to find the combination of antecedent conditions and design storms that consistently gives flows which match an observed flood frequency distribution. Since there are rarely enough data to define an observed flood frequency distribution, a synthetic flood frequency distribution may have to be used. This may be generated by continuous simulation (as above), or, as in the Flood Studies Report, by simulation in probability space (running the model with many different combinations of input variables sampled from their own probability distributions in order to build up a probability distribution of output). The advantage of the sensitivity analysis approach is that it may be repeated for several catchments to determine stable definitions for the inputs (design storm and antecedent conditions) which are applicable to each catchment. Once found, these inputs alone may be applied to new catchments, avoiding the need for simulation; the simulation is done at the model building stage, not at the model application stage. It was this type of approach that was adopted for the work described in this report.

An outline of the analysis

The analyses described in this report are somewhat involved. The search for a stable set of design inputs requires flow frequency data for several catchments. Unfortunately, long stationary records of such data do not exist. It was therefore necessary to generate synthetic flow records by passing a long-term rainfall record through a model (which in some cases had been fitted to a catchment where suitable rainfall-runoff data existed). Although it was not strictly necessary to use real catchments with observed data, it was considered advantageous to do so since the fidelity with which the model represents observed catchment behaviour could be investigated. Firstly, it was possible to check how well the model reproduced observed events; and secondly, the derived synthetic flood frequency distribution could be checked against the observed distribution at low return periods.

Five catchments were used in the analyses. Two of these (Stevenage and Derby) were existing catchments where some check with reality was possible. In addition, three imaginary catchments were used - ST1, ST2 and Stevenage (SW). All five catchments are described in section 3. Two rainfall records were used in the derivation of the synthetic flood frequency distributions for these catchments. One of these is representative of the south-east of England and was applied to Stevenage, Derby and ST1, while the other, representative of the wetter, south-west of England, was applied to ST2 and Stevenage (SW). Both records are described in section 4. Flood frequency distributions were produced for ten points within each catchment.

Once synthetic flood frequency distributions had been generated, the second stage of the analysis could commence. The model output to a range of design inputs was examined to find a set of inputs which would yield as close a match as possible to the synthetic flood frequency distributions previously derived.

Compared to the approach previously adopted for the Flood Studies Report (NERC, 1975) the present analysis differs in three main respects. Firstly, in the first stage of the analysis, a continuous simulation procedure (in the time domain) was adopted to generate the synthetic flood frequency distribution. This was because the Wallingford Model is much more complicated than the Unit Hydrograph model used in the Flood Studies Report and probability simulation (in the probability domain) requires a great many more model runs and correspondingly prohibitive computer costs. Furthermore, the probability simulation technique used in the Flood Studies Report requires the input variables to be statistically independent, a condition shown to be valid for floods in natural catchments but plainly invalid for urban catchments where severe rainfall events occur in the summer when catchments are dry (resulting in an inverse correlation between rainfall return period and catchment wetness). The second difference, in the second stage of the analysis, concerns duration. Because the sewer design model is a distributed model, requiring computations at all points within the system, a range of storm durations is considered within the model - the design duration being that which gives the largest discharge. Duration is thus not considered as an input variable to the model to be fixed *a priori*. The third difference, also in the second stage of the analysis, concerns equality of return period between rainfall depth and runoff peak. This generally-assumed equality is so deeply established in engineering practice that a stable set of inputs was sought which incorporated this constraint. As a result, rainfall depth was not considered as a variable in the sensitivity analysis.

Thus, definition of two variables remained to be found (storm profile and urban catchment wetness index,UCWI). An arbitrary profile was adopted (50% summer), and it remained to determine the value of UCWI which gave a closest match to the synthetic flood frequency distribution. The method by which this value of UCWI is obtained for a given catchment is described in section 5. A method for determining the design value of UCWI based on climatic circumstances is presented. Section 6 examines the sensitivity of the model to changes in the design inputs.

Section 7 reports on an examination of the Rational Formula based on the probability data generated in this study. Section 8 draws general conclusions from the work.

2. THE RAINFALL-RUNOFF MODEL

The design package being developed at the Hydraulics Research Station and the Institute of Hydrology comprises four basic models:

a. a peak flow model (c.f. the Rational Formula),

b. a peak flow model incorporating pipe-slope optimisation,

c. a hydrograph model for design and simulation (but including only single pipe surcharging), and

d. a hydrograph simulation model including full solution for surcharged flow.

The last model is the most detailed in its simulation of the rainfall-runoff process; and because model c. is effectively a special case of model d. it is the last one which has been used in this study. From here on, it is called the Wallingford Model.

The model may be divided into two:

a. an above-ground hydrological model (The Wallingford Urban Subcatchment Model), described in detail in a companion report (Kidd & Lowing, 1979) and incorporating

   i. a contributing area runoff volume submodel, and

   ii. a surface routing submodel comprising separate nonlinear reservoirs for paved/pervious areas and for roofed areas.

b. a below-ground hydraulic model, incorporating

   i. a Muskingum-Cunge pipe routing model for part-full flow (Price & Kidd, 1978), and

   ii. a full solution of the simultaneous differential equations for surcharged pipe flow (Bettess, Pitfield & Price, 1978).

Catchment data requirements are comparable to the RRL method. The basic space unit is from manhole to manhole; for each unit, the requirements are:

i. the area of paved, roof, and pervious surface;

ii. the average surface slope and area-per-gully, each entered as one of three categories (steep, medium or flat for slope; small, medium or large for area-per-gully);

iii. pipe length, slope, diameter and roughness; and

iv. manhole depth and area, together with a "floodable area" (for surcharge solution).

Global values for some of these inputs may be specified (eg, surface slope, area-per-gully, pipe roughness). Values for the surface and pipe routing parameters are normally calculated by the model from the above inputs, though the model may be constrained to adopt other values determined from local flow data.

The hydrological input to the model consists of a rainfall event (or, in design, rainfall depth, duration and profile) and a value for the catchment wetness index UCWI defined as:

$$UCWI = 125 + 8\ API5 - SMD \qquad (2.1)$$

where API5 is the 5-day antecedent precipitation index (mm)

and SMD is the soil moisture deficit (mm).

The overall catchment percentage runoff (PRO) is estimated from UCWI, and from the overall percentage imperviousness (PIMP) and soil-type (SOIL) according to the following relationship derived by regression analysis (Kidd & Lowing, 1979):

$$PRO = -20.7 + .829\ PIMP + 25.SOIL + .078\ UCWI \qquad (2.2)$$

The first three terms in this equation relate only to catchment characteristics, and represent what is called the standard percentage runoff (SPR):

$$SPR = -20.7 + .829\ PIMP + 25.SOIL \qquad (2.3)$$

Equation (2.2) can then be written:

$$PRO = SPR + .078\ UCWI \qquad (2.4)$$

Where no local flow data exist, equation (2.4) is used with SPR estimated from equation (2.3). If local data are available, a better estimate of SPR may be found by taking the mean observed value of PRO - .078 UCWI; where possible this has been done in this study.

3. CATCHMENT DATA AND MODEL FITTING

Introduction

As outlined in section 1, five catchments were used in the present study: two existing gauged catchments - Stevenage and Derby - where some check on model performance was possible; and three imaginary catchments - ST1, ST2, and Stevenage (SW) - where no such checks were possible. Within each of these catchments ten points were chosen as design points on which to perform the analysis. Thus, in all, 50 design points were used covering a range in catchment area from .2 ha to 134 ha, in slope from 1 in 29 to 1 in 165, and in percentage imperviousness from 18% to 61%. Details of the catchments and of model fitting follow.

The Stevenage catchment

Stevenage is a 142 ha catchment on a fairly pervious soil (SOIL = .3).

It is 23% impervious, with 796 pipes at an average slope of 1 in 38. It is rarely subject to surcharging. Since 1974 flow has been monitored in one pipe upstream of the outfall using an Arkon air-purge system. Some doubt existed over the performance of the Arkon meter at high discharges and in 1977 a flume structure was built at the outfall. Subsequent comparison of the Arkon and flume data has shown generally good agreement. There are two rainfall recorders on the catchment, but these are not logged together with the flow, so some timing discrepancies are present.

Details of the ten design points chosen from the Stevenage catchment are given in Table 1, along with the same information for the outfall (pipe 1.42) which was not used as a design point (for reasons discussed later).

TABLE 1: DESIGN POINT DETAILS FOR STEVENAGE CATCHMENT

Pipe	Total catchment area (ha)	Paved area (%)	Roof area (%)	Average pipe slope (1 in)
1.42	142	13	11	38
1.41	132	14	12	37
1.37	102	15	12	36
1.33	64	12	11	35
1.14	23	10	8	36
113.30	29	21	15	36
113.23	27	21	15	37
113.14	6.6	19	13	33
113.12	3.7	14	12	33
31.14	18	13	9	37
194.29	26	9	8	50

The Wallingford Model was fitted to Stevenage using area and pipe data only. To test the fit, several storms for which rainfall and high-quality flume data were available were run through the model. Figures 3.1 to 3.3 demonstrate the typical fit of modelled to observed hydrographs. For these storms, percentage runoff was "forced" such that the volumes of modelled and observed runoff were equal. The storms were of reasonable severity, those of Figures 3.1 and 3.3 each corresponding to about a half year return period. It must be emphasised that no flow data (other than the volume of runoff) have been used in fitting the model, and it was considered that the fit was good enough not to try improving on it by using locally derived routing parameters.

The fit of the model to individual events has been verified using the observed percentage runoff. To use the model in simulation, the percentage runoff equation given as equation (2.4) was used, ie:

$$PRO = SPR + .078\ UCWI \qquad (3.1)$$

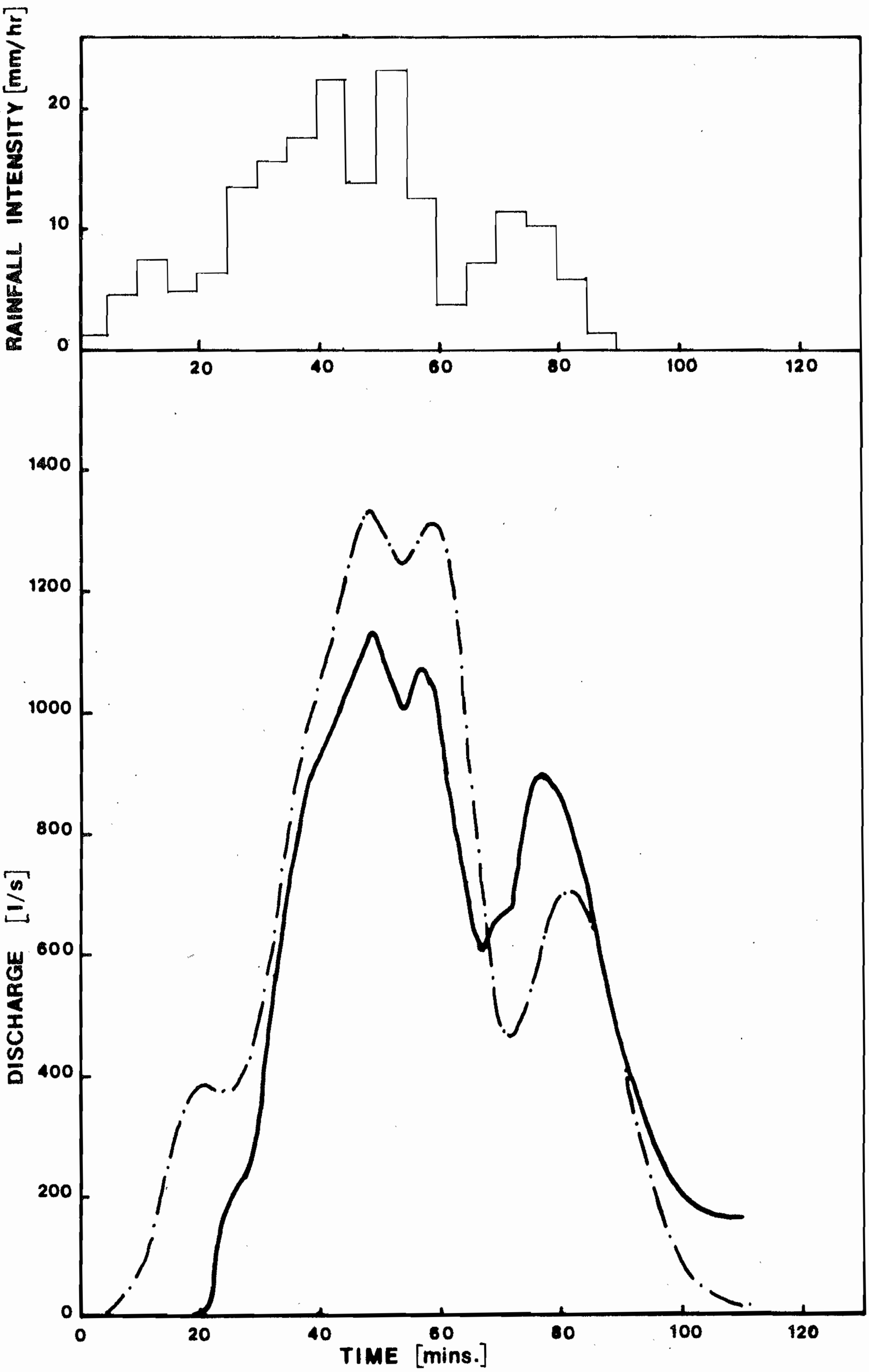

FIG 3.1 STEVENAGE : STORM 6/8/77

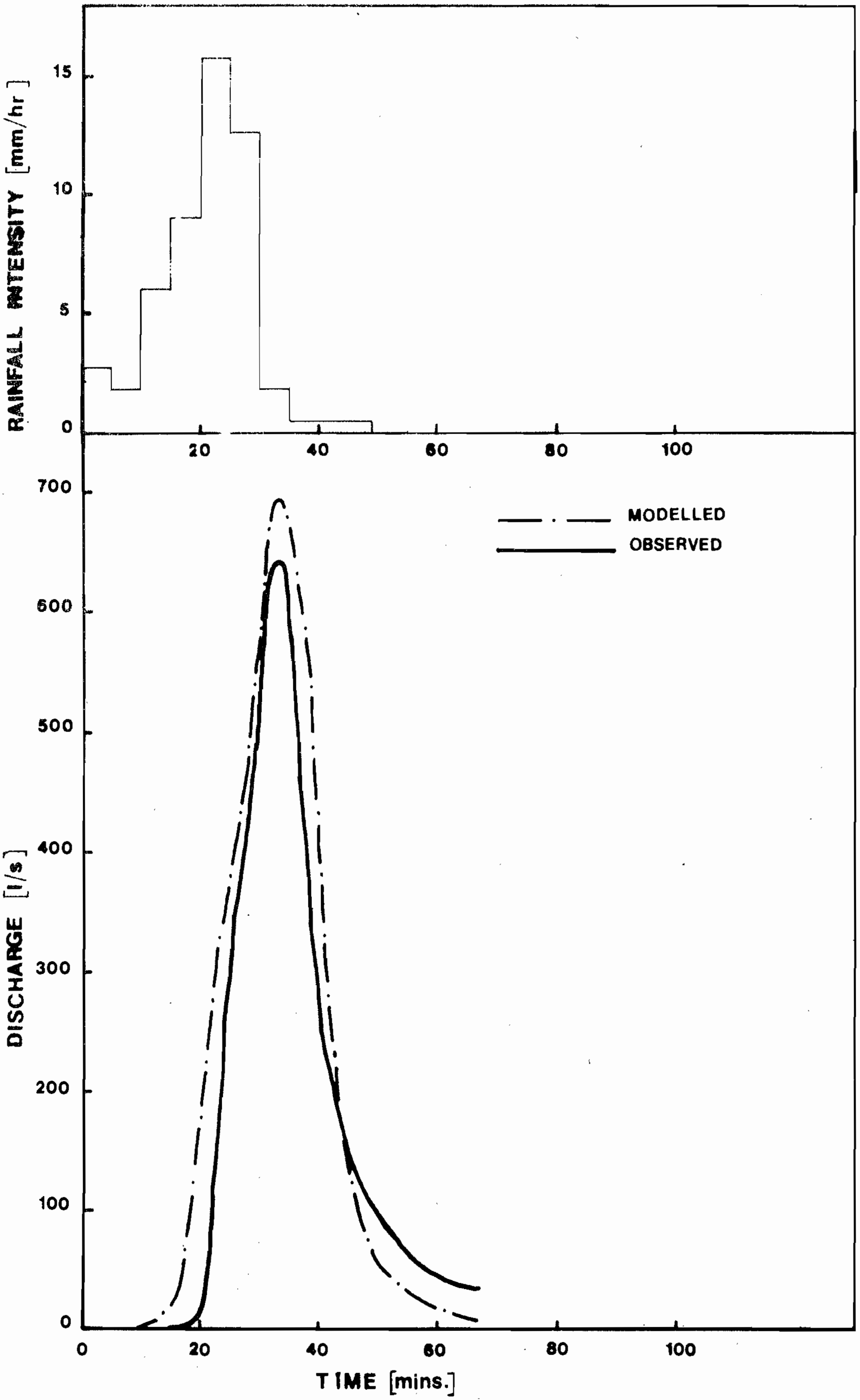

FIG 3.2 STEVENAGE STORM OF 26/8/77

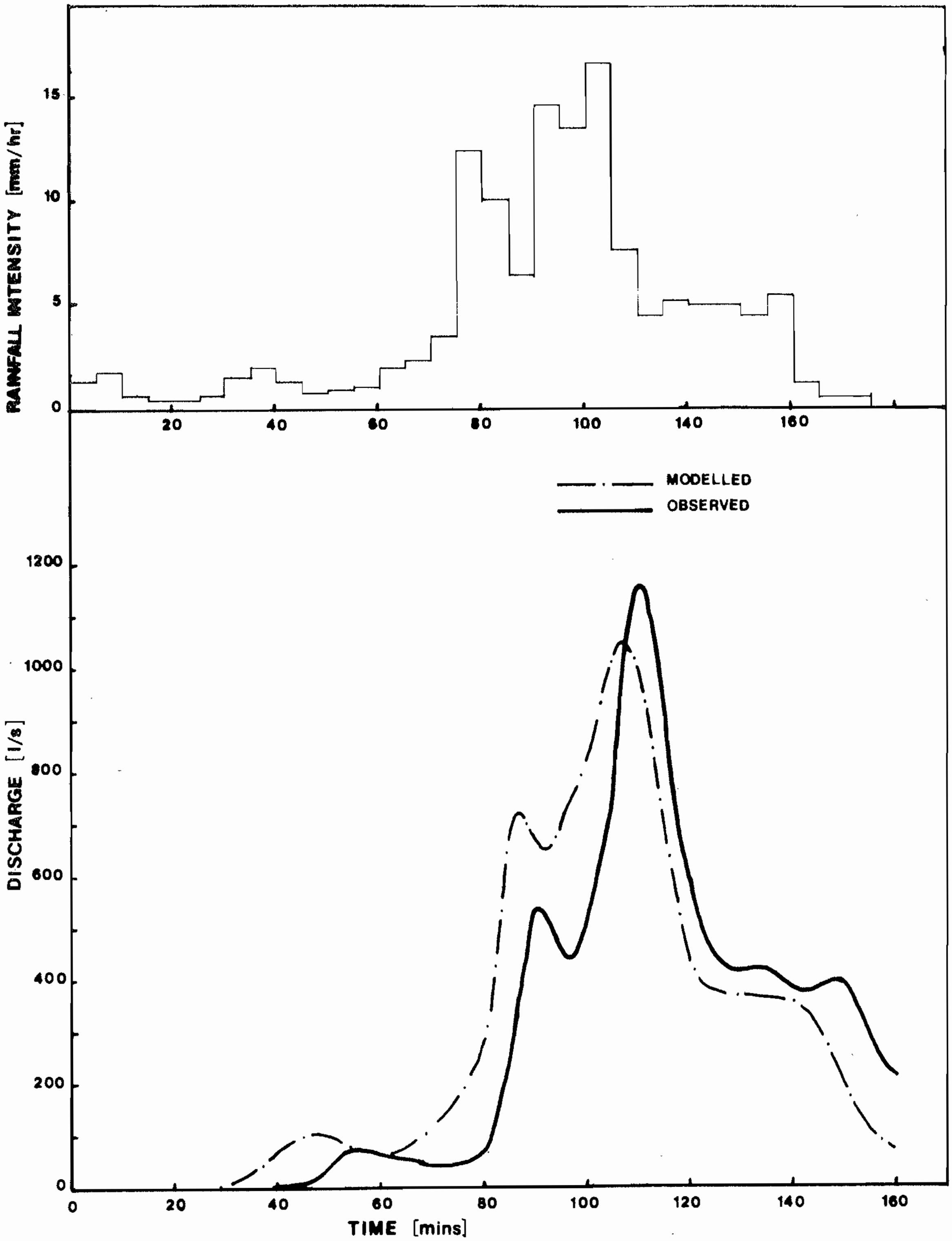

FIG 3.3 STEVENAGE STORM OF 21/10/77

Equation (2.3) predicts a value of SPR for Stevenage of 6.20. However, following the procedure outlined in section 2, equation (2.4) was fitted to observed events from the Arkon meter record. The locally derived value obtained for SPR was 7.78. Equation (2.4), thus fitted, was used in generating the synthetic flood-frequency distribution. The Arkon data was preferred to the flume data for fitting equation (2.4) since a longer record was available; any errors in gauging peak flows would not greatly affect volume estimation. However, the Arkon gauge refers to pipe 1.41, and not the catchment outfall. For this reason, and to allow a comparison of the four years of Arkon record with the synthetic flood-frequency distribution, pipe 1.41 and not the outfall was used as the design point in subsequent analysis.

The Derby catchment

Derby is a 10 ha catchment on a fairly impervious soil (SOIL = .45). It is 53% impervious (ie, paved surfaces), with 87 pipes at an average slope of 1 in 87. It is subject to frequent surcharging. Since 1972, flow has been monitored at three points in the system using an Arkon air-purge system. The depth-discharge relationships have been calibrated at low flows using dilution gauging. Backwater effects from downstream have been identified for many of the larger storms on record. The nearest raingauge is situated about one km from the catchment, and timing discrepancies are present.

Ten design points were chosen from the catchment, including the three gauged sites (pipes 1.26, 1.23, and 1.17). Details are given in Table 2.

TABLE 2: DESIGN POINT DETAILS FOR DERBY CATCHMENT

Pipe	Total catchment area (ha)	Paved area (%)	Roof area (%)	Average pipe slope (1 in)
1.26	10.3	37	16	87
1.23	8.5	34	17	82
1.17	7.2	31	18	92
1.15	5.3	28	16	67
1.07	2.7	28	17	49
1.05	1.8	33	7	34
6.05	1.7	38	23	165
17.04	0.2	28	31	29
18.04	0.3	21	22	100
18.09	1.6	45	16	99

Once again the Wallingford Model was fitted using area and pipe data only and the fit tested by running several storms through the model. Figures 3.4 to 3.6 demonstrate the typical fit of modelled to observed

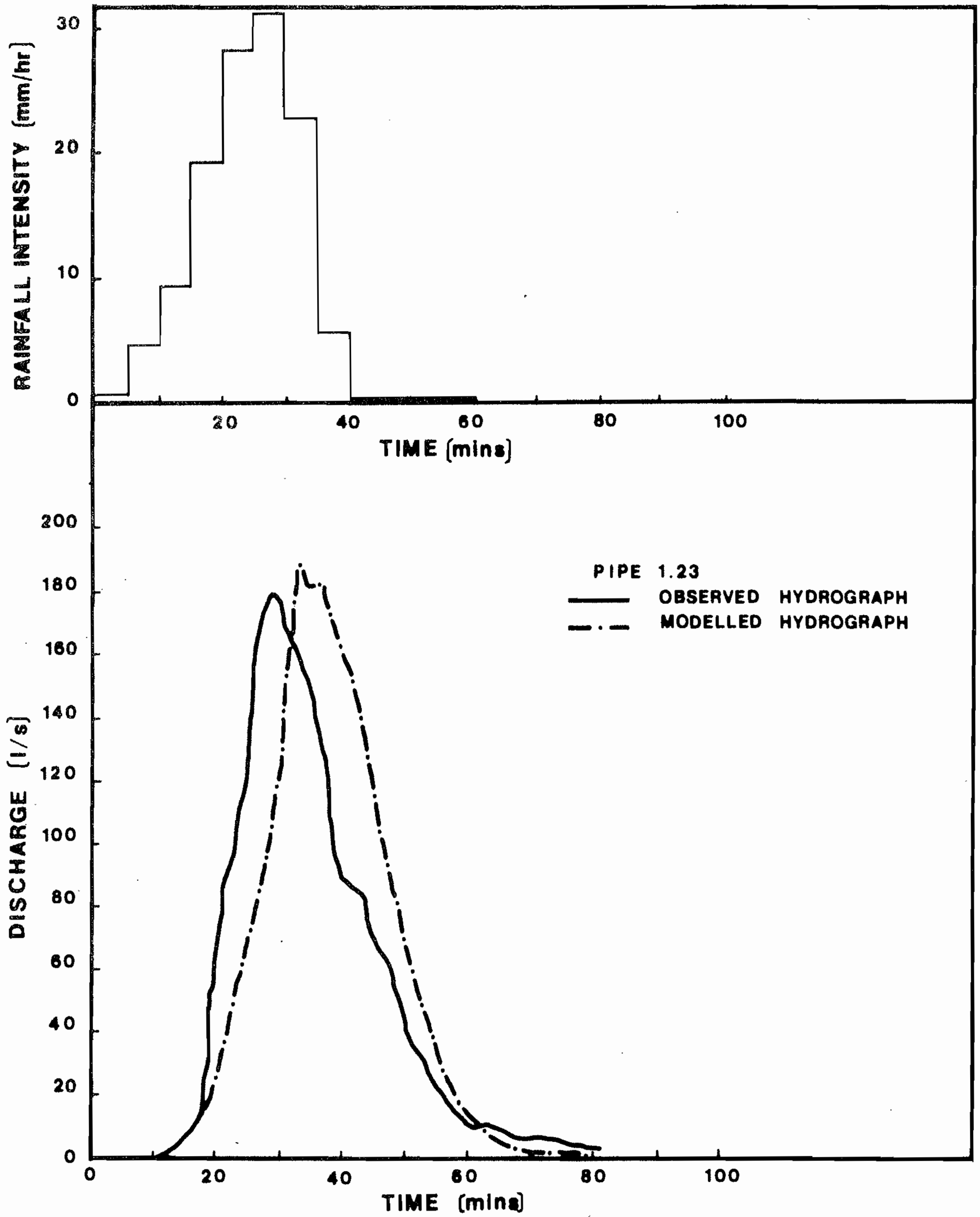

Figure 3.4 DERBY: STORM 31/7/72

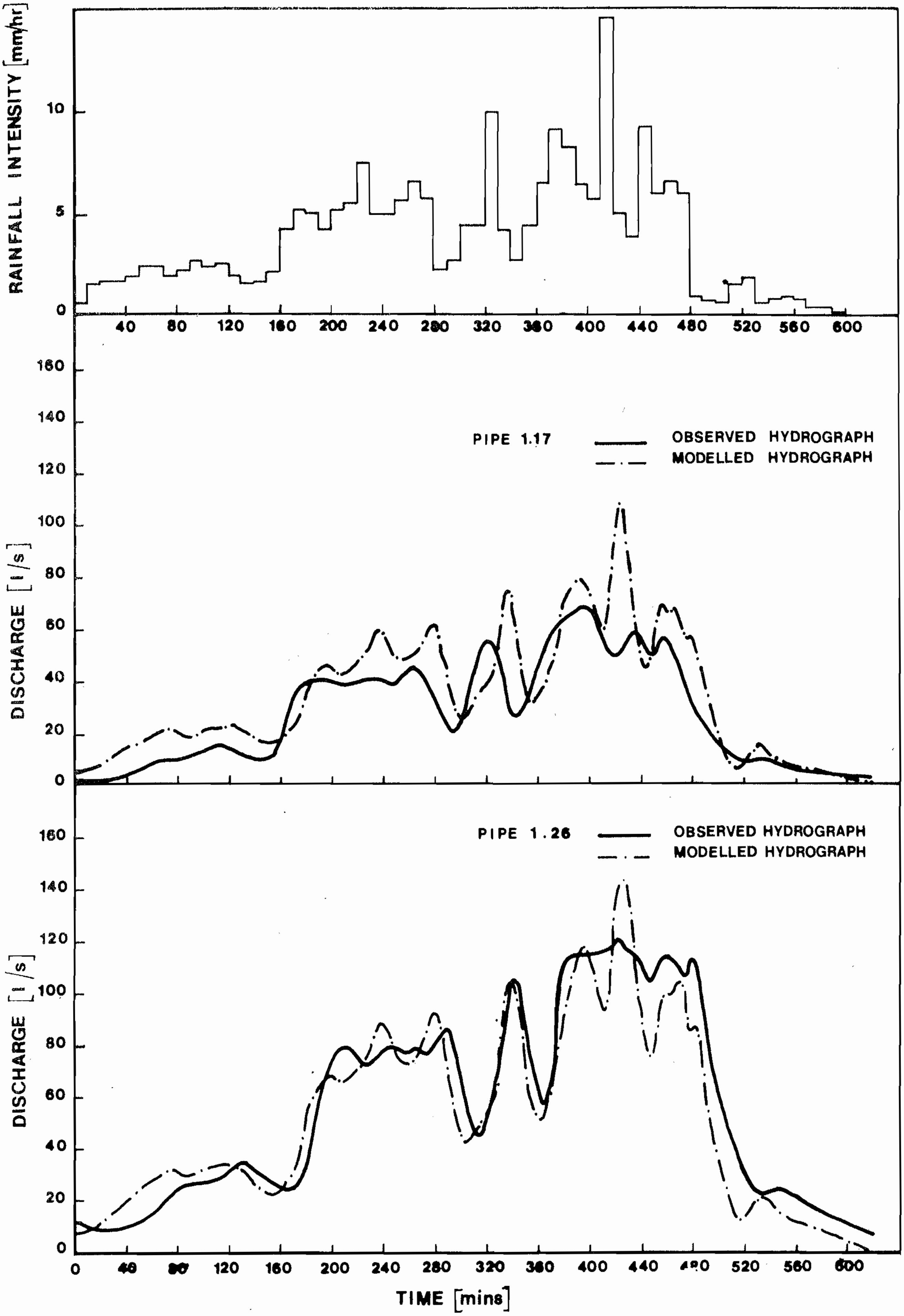

Figure 3.5 DERBY: STORM 8/9/72

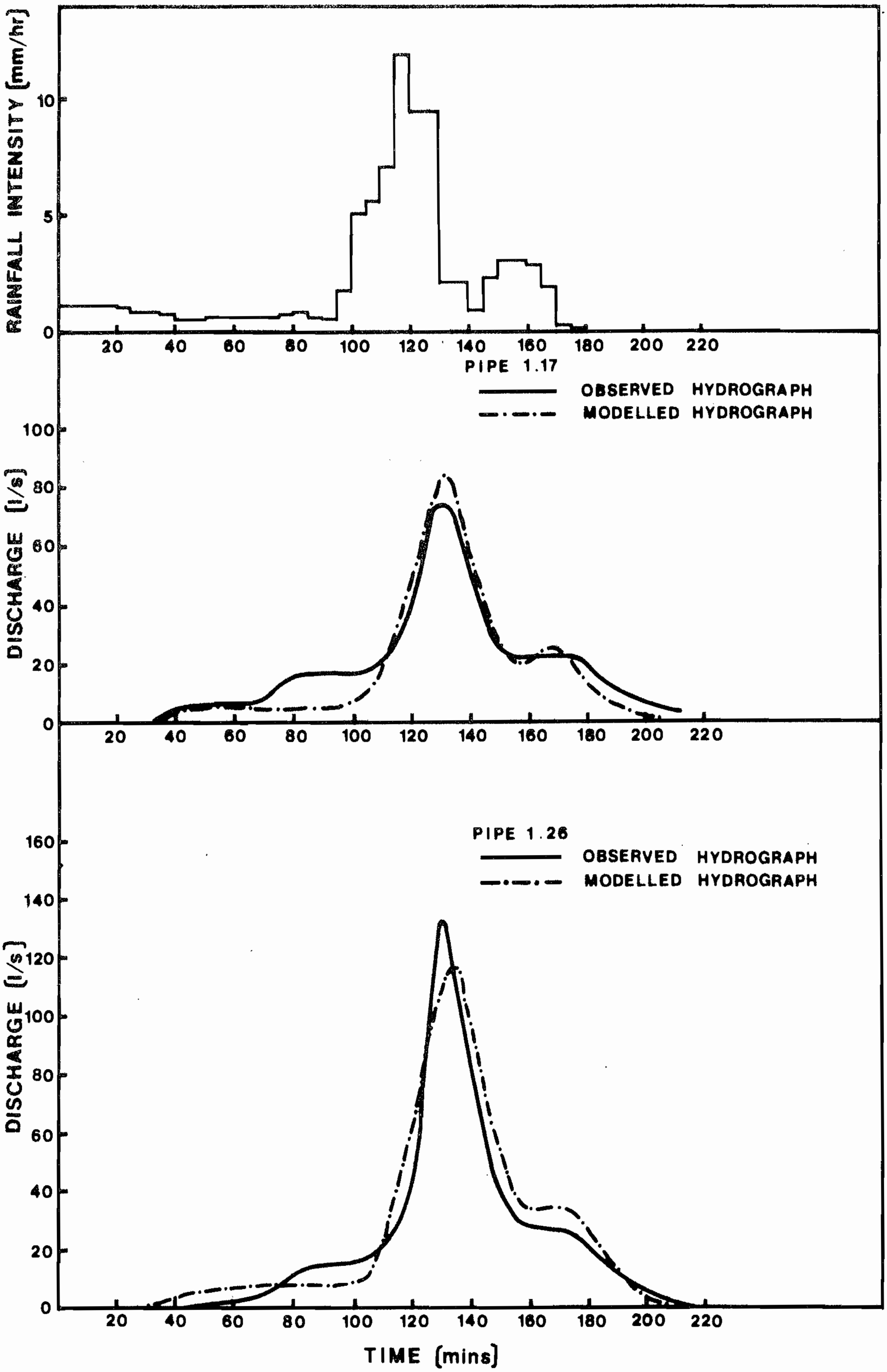

Figure 3.6 DERBY: STORM 12.2.73

that was obtained. Again percentage runoff was "forced", but this time based on an average of all three gauges. The storm of Figure 3.4 is the largest for which reliable data were available, corresponding to an observed return period of about 1.5 years. The storms of Figures 3.5 and 3.6 have each return periods of about 0.25 year. Once again it was thought that the fit of the model would not be improved by using locally-derived routing parameters.

To obtain the best-fit SPR for use in the percentage runoff equation (2.4), the data for pipe 1.26 was used. This gave a locally-derived value for SPR of 37.0 compared with the value from equation (2.3) of 34.5. Equation (2.4) thus fitted was used in generating the synthetic flood-frequency distribution.

Artificial catchment, ST1

The Stevenage and Derby catchments were thought to represent extreme conditions, Stevenage being relatively steep with a low percentage imperviousness, Derby being very flat with a high percentage imperviousness. Consequently an artificial catchment (ST1) was designed to be intermediate between these two. This catchment was of the same basic form as Stevenage, having the same pipe layout and the same soil type, but pipe slopes were all halved, and all roof and paved areas increased by the factor 1.8. Pervious areas were correspondingly reduced in order to yield the same total catchment area as Stevenage (142 ha) but with an increased percentage imperviousness of 42%. Substituting these values for SOIL and PIMP into equation (2.3) gave a value for SPR of 21.7, approximately half way between that for Stevenage and Derby. Pipe sizes for ST1 were designed by the Wallingford Model using a 30 minute storm of 50% summer profile and a depth of return period two years for S.E. England. Design UCWI was chosen as 50. Although the results of this study might suggest other design input, the use of the above input in no way compromises the results of ST1; ST1 was designed to have a realistic sewer system such as might be typical of any real catchment.

Design points for ST1 were chosen at the same points in the sewer system as for Stevenage and details are given in Table 3.

TABLE 3: DESIGN POINTS FOR ST1 CATCHMENT

Pipe	Total catchment area (ha)	Paved area (%)	Roof area (%)	Average pipe slope (1 in)
1.42	142	23	19	76
1.41	134	24	20	75
1.37	106	26	21	71
1.33	64	22	20	70
1.14	22	19	16	71
113.30	32	33	24	73
113.23	31	33	23	73
113.14	7.1	32	21	67

TABLE 3: (continued)

Pipe	Total catchment area (ha)	Paved area (%)	Roof area (%)	Average pipe slope (1 in)
113.12	3.8	24	21	67
31.14	18	23	17	75
194.29	25	17	16	100

Artificial catchments ST2 and Stevenage (SW)

The three catchments described so far were each situated in the same rainfall regime typical of S.E. England. In order not to bias the results of this study to this rainfall regime, the analysis was repeated for catchments in the wetter western part of England. Unfortunately, no long flow records were available for sewered catchments in these areas, so the model fit to observed flood events and observed flood-frequency distributions could not be tested.

However, since the Wallingford Model was shown to represent catchment behaviour adequately for S.E. England catchments, it was considered permissible to investigate the effect of a different rainfall regime using artificial catchments. Consequently, catchments ST1 and Stevenage were transposed to S.W. England and called ST2 and Stevenage (SW) respectively. All catchment details for ST1 and ST2, and for Stevenage and Stevenage (SW) were identical except that pipe sizes were re-designed using the Wallingford Model for rainfall depths of two year return period expected in S.W. England. The same design points were chosen from each catchment, details are the same as for their corresponding S.E. England counterparts (for Stevenage (SW) see Table 1, for ST2 see Table 3).

4. HYDROMETEOROLOGICAL DATA

Introduction

As was described earlier, the method adopted for generating the flood frequency distribution was to pass a long record of rainfall and soil moisture deficit through the model (after fitting). The necessary one-minute interval rainfall data was required to be representative of the climatic regime of the catchment in question. Two such sequences were supplied on magnetic tape by the Meteorological Office at Bracknell. These data had been obtained from the PEPR system (Folland & Colgate, 1978) currently being used for the digitisation of UK autographic rainfall data. These two sequences will be described separately.

The S.E. England Rainfall Series

A 98-year rainfall series, considered to be representative of the climatic regime of the two main catchments in the study (Stevenage and Derby), was formed by putting three individual stations' records together, end to end. Details of these three stations, with appropriate parameters, are given in Table 4 (where SAAR = Standard Average Annual Rainfall (1941-70), M52D = two day rainfall depth of five year return period, r = ratio of one hour to two day rainfall depths of five year return period).

TABLE 4: STATION DETAILS FOR S.E. ENGLAND RAINFALL SERIES

Station	SAAR (mm)	M52D (mm)	r	Years of record
Farnborough	725	55.0	.365	1941-74
Abingdon	600	46.5	.405	1954-75
Hampstead	670	49.0	.435	1941-75
Mean	665	50.0	.40	98 years

While monthly rainfalls for these three stations would probably be correlated, for the short duration rainfall events of the type of interest here the stations may be considered independent. An inspection of the end of month SMDs for the three stations showed that these also could be considered independent. The stations are approximately 40 miles apart. The mean values given in Table 4 were assumed to be representative of the full 98 year data set. However, as will be shown later, the rainfall series did not exactly conform to the Flood Studies Report (FSR) model with respect to these mean values.

The data was set up as a series of daily (9 am to 9 am) blocks. Each block had a header which gave the date, SMD at 9 am and rainfall volume for the day in question. Where this volume was greater than zero, cumulative rainfall was then given as 1440 integer values in 1/100ths of a millimetre. In this way, long spells of zero rainfall were excluded from the data.

Prescreening the data

With the data in the form described above, it was still much too lengthy for running through the model. Consequently a pre-screening analysis was used to abstract all the rainfall events which could possibly contribute to a significant flood. For this analysis, a method of defining an individual event was required, and also some method of deciding whether that event might yield a significant flood. Based on a pilot study, the following event definition was chosen : an event starts with the first non-zero rainfall ordinate, and ends when the rainfall intensity falls below a threshold figure of 1 mm/hr for a 15 minute period. Thus, a period of rainfall in which the intensity falls below 1 mm/hr for 15 minutes may be split into two events. This definition is necessarily arbitrary but, from experience, it is one which will produce runoff peaks which may be considered to be independent. This last criterion is important, as will be seen later. Once an independent event has been identified, it is then considered to be significant if the rainfall return period in any critical duration within the event between five minutes and two hours is in excess of half a year. Using the above definitions, the data for days on which one or more significant events occurred were abstracted and stored on a shortened file. An index of significant events was maintained. An example of part of the output from this prescreening programme is shown in Table 5.

TABLE 5: EXAMPLE OF PRESCREENING PROGRAM OUTPUT

SEQ.	DATE	TIME	Rf VOL (mm)	DUR. (min)	RETURN PERIOD IN CRITICAL DURATION (yrs)							SMD	API5	UCWI
					5 MIN	15 MIN	30 MIN	45 MIN	60 MIN	90 MIN	120 MIN			
1	14.09.54	01.58	5.2	34	.55	.26	.22	.15	.12	.08	.06	0	12.56	225.5
2	14.09.54	06.57	8.6	32	.89	.77	.61	.58	.44	.29	.22	0	21.51	297.1
3	24.11.54	07.58	31.6	361	.24	.22	.36	.38	.46	.84	1.28	0	3.21	198.7
4	06.06.55	19.46	11.5	50	.78	.74	1.39	1.23	.97	.65	.49	0	.03	125.2
5	18.07.56	12.17	7.7	65	.39	.67	.45	.37	.31	.21	.16	79.2	4.23	79.7
6	02.08.56	5.10	26.0	386	.11	.13	.16	.17	.19	.32	.58	85.5	.00	39.5
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The calculation of the return period of the rainfall in any critical duration was calculated from the Bilham Formula (Bilham, 1935) which is given by:

$$T = \frac{.00494\ (R + 2.54)^{3.55}}{D} \qquad (4.1)$$

where T is the return period in years,
R is the rainfall volume in mm,
and D is the duration in minutes.

The Bilham model can be compared with the Flood Studies model (NERC, 1975) to give a better estimate of the threshold return period. In fact, the Bilham model agrees quite well with the Flood Studies model in S.E.

England (at low return periods and short durations), and the 0.5 year return period Bilham threshold is approximately a 0.45 year threshold according to the Flood Studies model.

The prescreening program extracted a total of 319 events from the 98 years. The justification of the prescreening process is based on the fact that, although these events will not produce the 319 largest floods in the record, the largest, say, 98 floods may safely be assumed to come from this prescreened record.

Examination of the rainfall record

Some interesting insights are possible from an examination of some of the features of these prescreened events. Firstly, with respect to the seasonality of the process, Figure 4.1 shows a frequency distribution of the 319 events on a month to month basis. This demonstrates that the rainfall season for these short duration events is very definitely a late summer one, very few significant events being observed in the first four months of the calendar year. Figure 4.1 also shows the seasonal distribution of the 98 largest floods (for one of the catchments) resulting from these rainfall events. This shows that the seasonal nature of the problem has become even more marked, and that over half the events come from July and August alone.

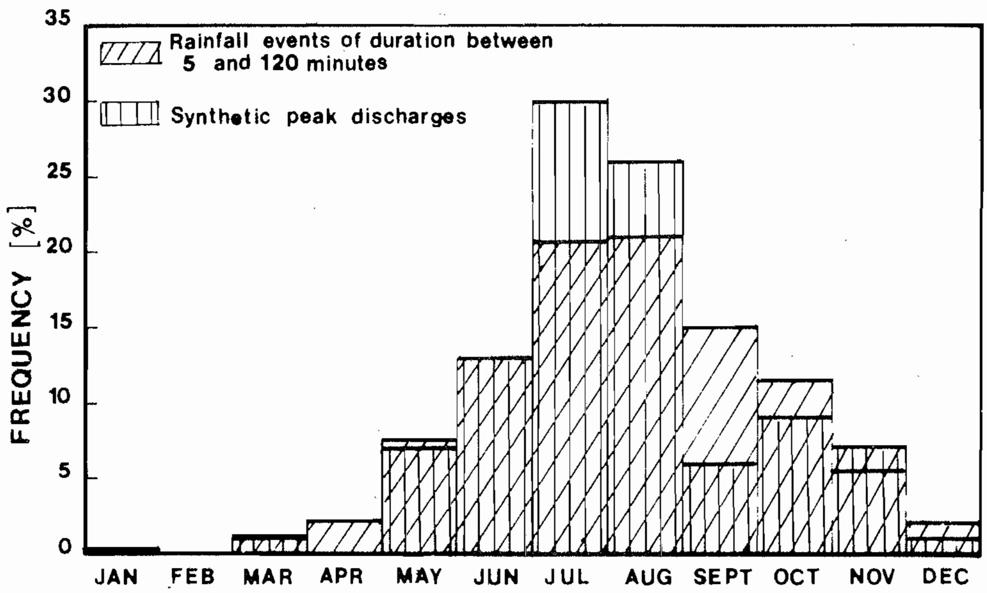

Figure 4.1 SEASONAL FREQUENCY DISTRIBUTION OF EVENTS FOR S.E. ENGLAND RAINFALL SERIES

Examination of the distributions of catchment wetness is shown in Figure 4.2 (this is also for the 98 largest floods rather than the whole data

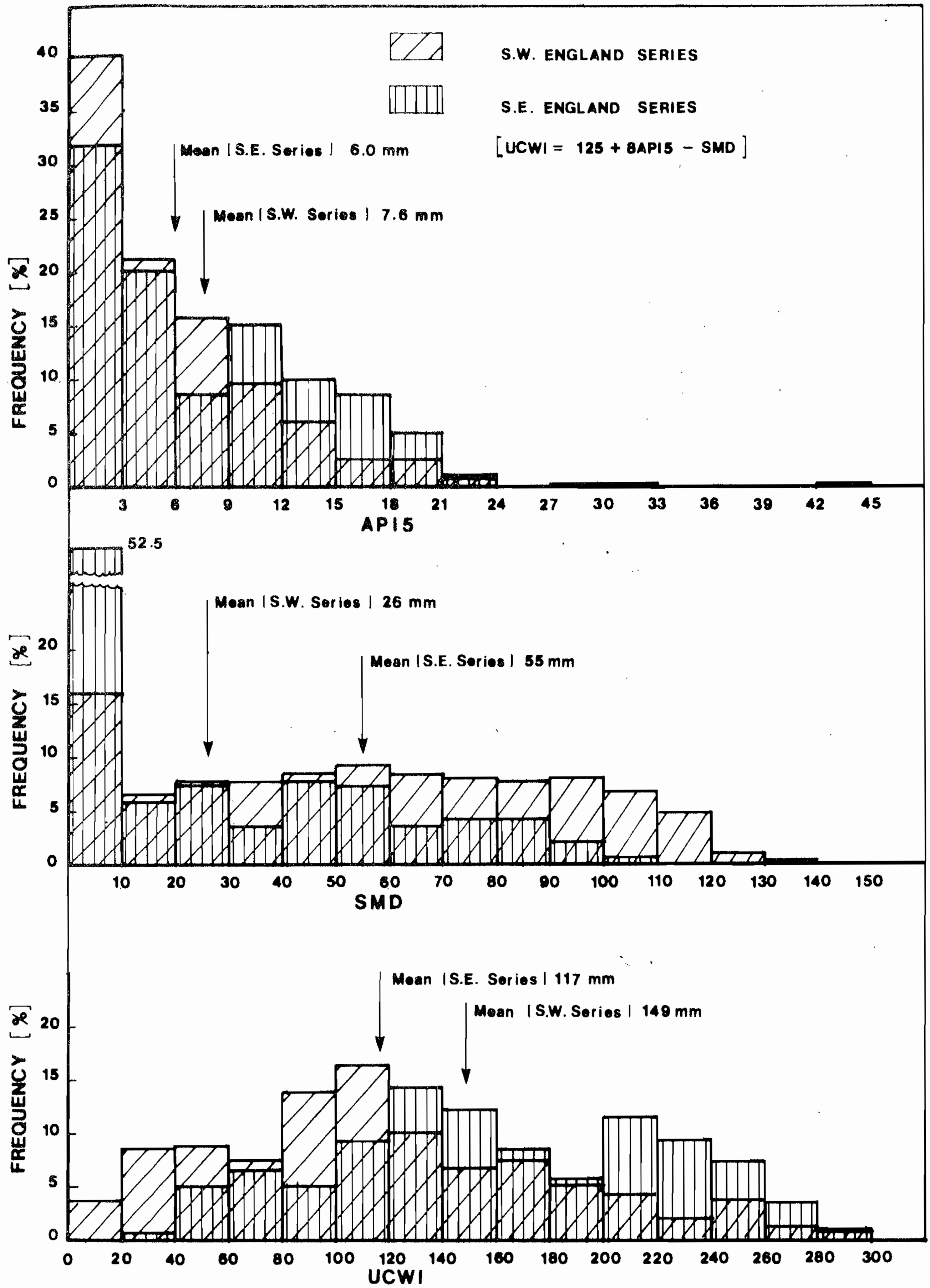

Figure 4.2 FREQUENCY DISTRIBUTIONS OF CATCHMENT WETNESS

of UCWI should vary with SAAR to reflect this phenomenon. Consequently frequency distributions of observed UCWI have been derived for various locations in the UK, and a relationship between the expected value of UCWI and SAAR has been determined.

UCWI is defined from soil moisture deficit (SMD) and 5-day antecedent precipitation index (API5) by the following equation:

$$UCWI = 125. + 8.\ API5 - SMD \qquad (4.3)$$

SMD is estimated daily at a number of sites in the UK by the Meteorological Office, Bracknell. The estimates are based on a running balance of rainfall and evapotranspiration calculated using the Penman equation. End of month values of SMD are tabulated separately for each site. In the Flood Studies Report (NERC, 1975) it was found that, in spite of the correlation between rainfall and SMD on an annual basis, if the data were considered on a seasonal basis, end of month values of API5 and SMD were independent, and furthermore, were representative of values experienced on days with heavy rain. Consequently, end of month values of UCWI could be used to represent the full UCWI distribution within any one season. In the present study, since urban flooding is shown to be essentially a summer occurrence (see Figure 2.4) a UCWI distribution was determined for the summer season only (months June to September).

From the SMD stations available, 27 were chosen to cover England and Wales, both geographically and in terms of SAAR. A map showing the location of the stations is given in Figure 4.6. For these stations, end of month UCWI values were determined for the years 1961-70. From these observed distributions, mean (expected value) median (middle value), standard deviation and interquartile range were abstracted and plotted against the stations' SAAR values (for the period 1941-70).

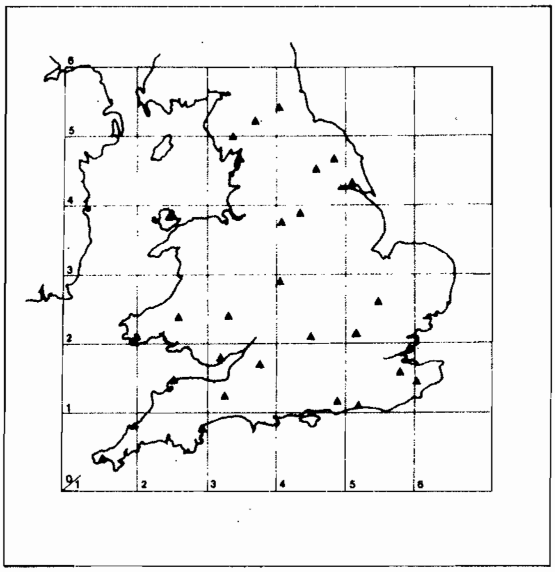

Figure 4.6 LOCATION OF SELECTED S.M.D STATIONS

The results for mean and standard deviation are presented in Figures 4.7 and 4.8. Although results for the mean do not depart far from a straight line, a curve is preferred, particularly when Figure 2.7 is compared with the corresponding relationship for natural catchment wetness index used in the Flood Studies Report. A weak trend towards higher standard deviation with higher SAAR (and thus higher mean UCWI) is observed.

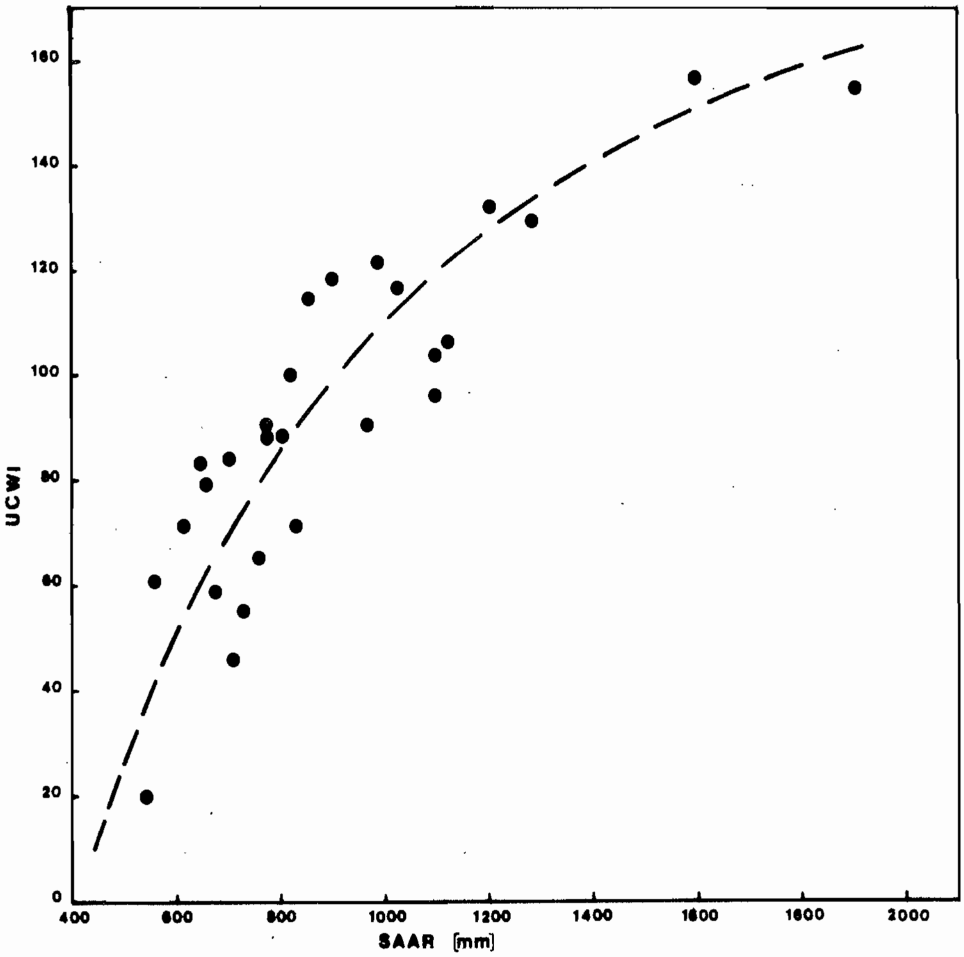

Figure 4.7 MEAN UCWI AGAINST SAAR

Several objective and subjective trend lines have been fitted to the data. The best unweighted least squares fit to the mean was a hyperbola giving a standard error of estimate of 15.4. However, a slightly different hyperbola given by:

$$\overline{UCWI} = 233 - 1.51\times10^{5}/(SAAR + 237) \tag{4.4}$$

with a standard error of estimate of 15.5 was preferred on subjective grounds. No attempt was made to derive a standardised frequency distribution for UCWI, but the above relationship for $\overline{UCWI}$ against SAAR was used later in the definition of design values for UCWI.

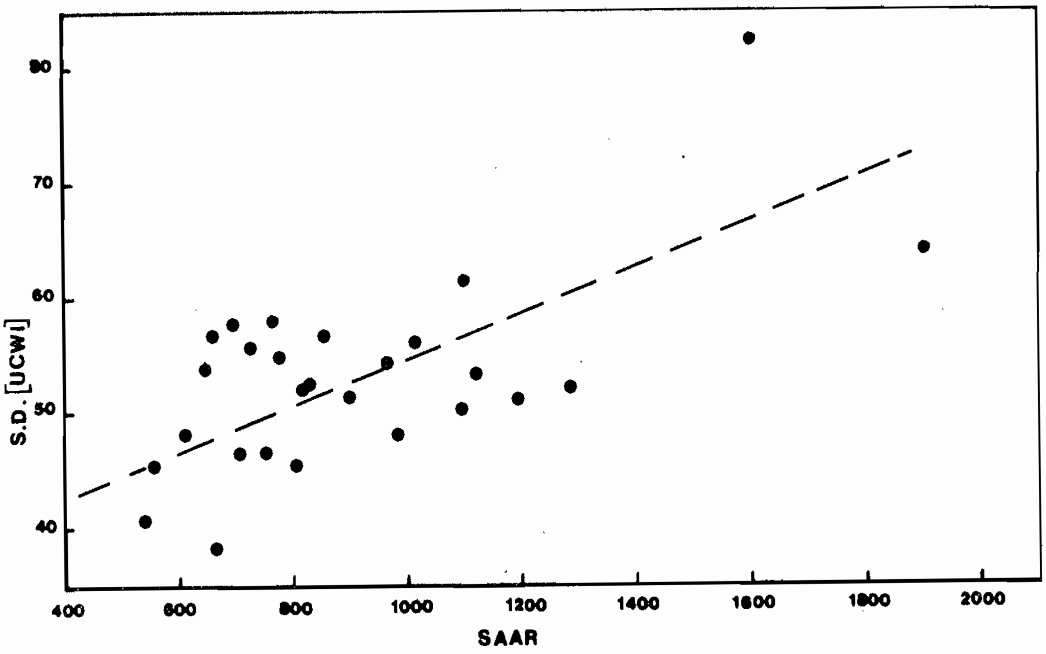

Figure 4.8 STANDARD DEVIATION OF UCWI AGAINST SAAR

5. THE ANALYSIS

Synthetic growth curve for Stevenage

The S.E. England rainfall series was used for the generation of the flood frequency distribution. As described in section 3, the runoff volume submodel was fitted to observed data from the Stevenage catchment, giving:

$$PRO = 7.78 + .078\ UCWI \qquad (5.1)$$

The standard error of the observed data about equation (5.1) was ±5%. A random noise component (of mean 0%, standard deviation 5%) was added back into the percentage runoff estimation to reproduce the observed variability. Using this relationship, the significant events abstracted from the S.E. England rainfall series were input to the model to give 319 flood events. As suggested in section 2, the analysis is based on the premise that the largest 98 flood events produced by this analysis would be the 98 largest floods in the 98-year record. A partial-duration series analysis was performed on these data with a threshold such that an average of one event per year was selected (ie, the annual exceedance series). The results of this analysis for the outfall of the Stevenage catchment (pipe 1.41) is shown in Figure 5.1 where each flood has been plotted against its exponential reduced variate, y; y is related to return period T by y = ℓn T.

Four years of data (1975-78) were available for pipe 1.41, so that some check with reality was possible. A partial-duration analysis of these data yielded the points plotted in Figure 5.1. This check showed that the synthetic flood frequency curve was in good agreement with observed data at low return periods. Besides the outfall, flood-frequency curves were also found for nine other points in the system. While this effectively repeats the analyses for nine further catchments, a consistency is found between different parts of the same catchment, as will be seen later.

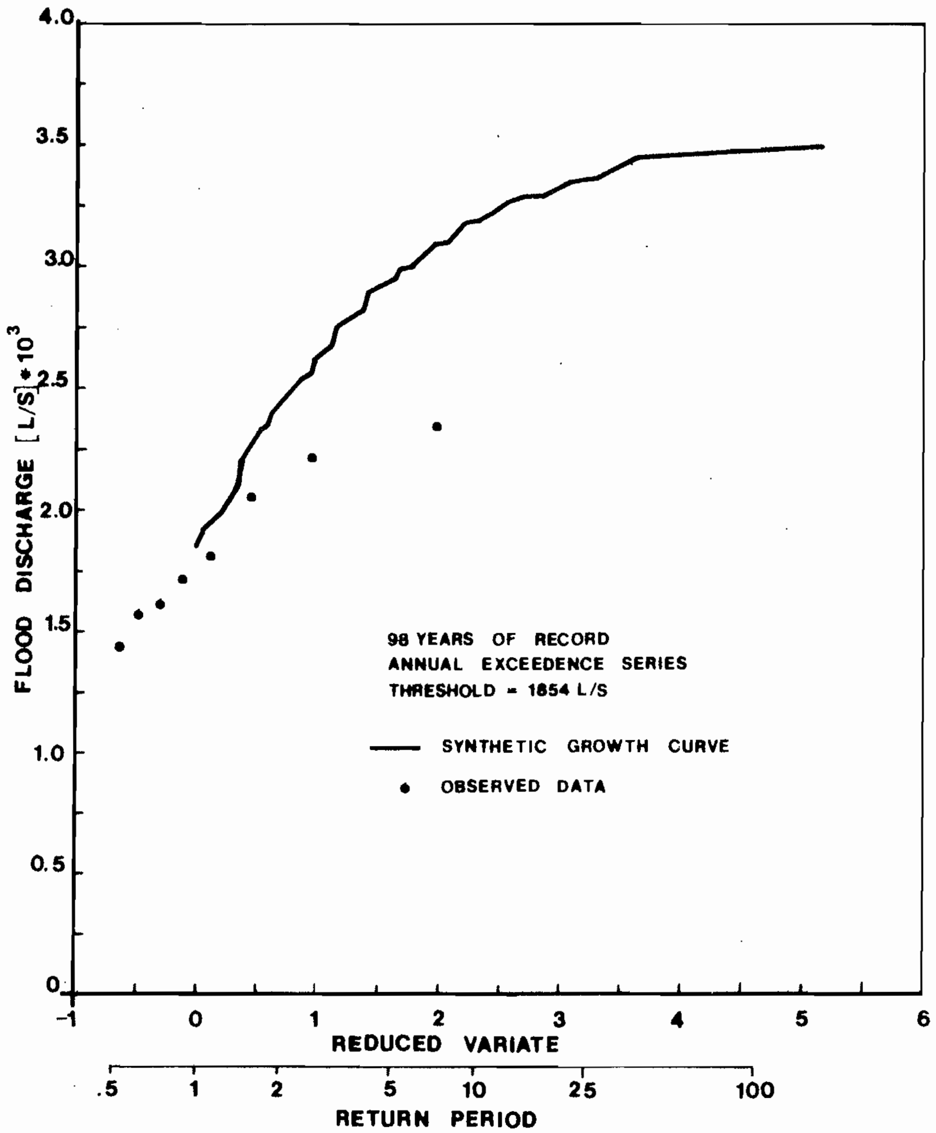

Figure 5.1 FLOOD FREQUENCY ANALYSIS FOR PIPE 1.41 AT STEVENAGE

Selection of best UCWI for Stevenage

Using the synthetic flood frequency relationships derived above, the next step was to examine the sensitivity of the design inputs to derive a set of inputs which would, on average, reproduce these relationships with the least error. Two of the four inputs were constrained: firstly, the return period of the rainfall was chosen to equal the required return period of the peak runoff; and secondly, it was decided to input storms of a number of durations (in fact, 15, 30, 60 and 90 minutes) and to take whichever of these produced the maximum peak runoff. There only remained, therefore, design values to be chosen for the rainfall profile and the catchment wetness index UCWI.

The range of possible profiles (or time distribution of the rainfall) has been taken from the Flood Studies Report, Vol II (NERC, 1975). The 50% summer profile is that which can be expected to be exceeded in terms of peakedness 50% of the time, this statistic having been derived from the analysis of summer storm events. Profiles have been published for 10%, 25%, 50%, 75% and 90% summer peakedness. These may simply be seen as labels since, unlike the other variables, these profiles will never actually be observed. Preliminary analyses showed that, as might be expected, the output of the design model is very sensitive to the profile, and it was found that the 50% summer profile was most appropriate. The synthetic and designed flood frequency distributions could not be matched for sensible values of UCWI for either the 25% or the 75% profile (the first giving floods which were too low and the second which were too high).

Thus the analysis was reduced to the definition of a value of UCWI which best matched the synthetic flood frequency curve. Simulations were performed for a range of return periods and UCWIs to produce a number of tables. An example of these tables is shown in Table 7 which represents the analysis for pipe 1.41 (the 30 minute duration proved to be critical for this point in the system). The flood discharges derived from the synthetic flood frequency distribution is also included in the last column of this table. These results are demonstrated graphically in Figure 5.2.

Scrutiny of Figure 5.2 and Table 7 shows that there is a systematic tendency for the required UCWI to decrease with increasing return period. This phenomenon is observed for all catchments, although the degree to which the required UCWI varies changes from one catchment to another.

There are three ways of accommodating this effect:

i. to allow the return periods of rainfall and runoff to vary systematically,

ii. to generate a relationship between design UCWI and return period, and

iii. to calibrate the method over a limited range of return periods (say, < ten years) and to introduce an allowance for the resulting over-estimate for high return periods at a later stage.

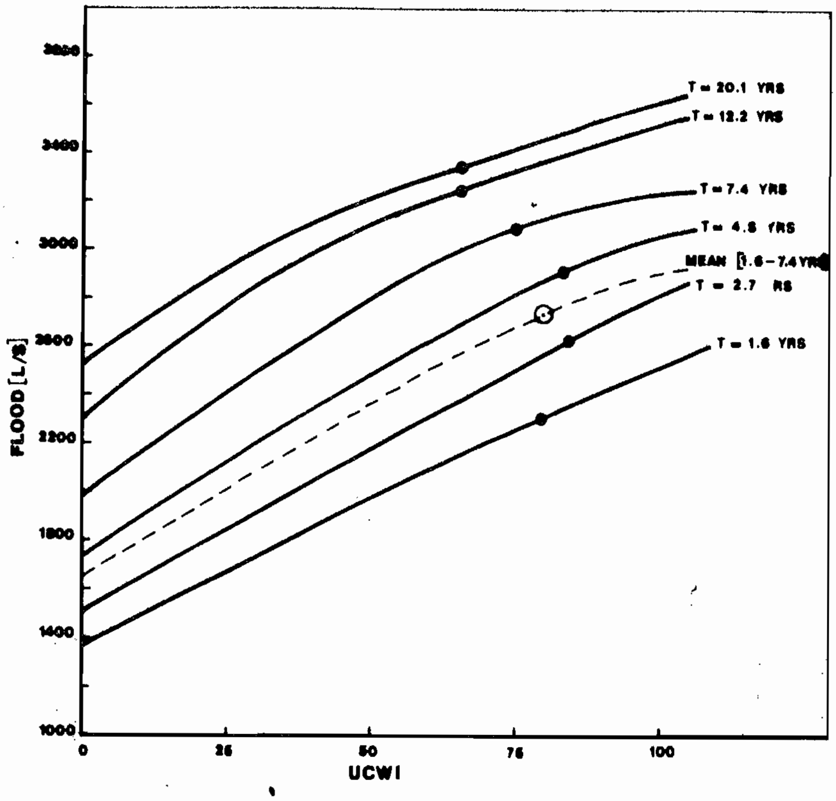

Figure 5.2 VARIATION OF FLOOD DISCHARGE WITH RETURN PERIOD AND UCWI FOR PIPE 1.41 AT STEVENAGE

TABLE 7: TABLE OF FLOODS FOR PIPE 1.41 AT STEVENAGE

Return period (Years)	Plotting position Y	Catchment wetness (UCWI) 0	25	50	75	100	125	150	Synthetic growth curve
1.0	0	-	-	-	-	-	-	-	(1856)
1.6	.5	1368	1667	1986	2261	2516	-	-	2300
2.7	1.0	1514	1848	2193	2497	2808	-	-	2626
4.5	1.5	1730	2132	2482	2814	3054	-	-	2908
7.4	2.0	1982	2416	2815	3085	3222	-	-	3091
12.2	2.5								(3240)
20.1	3.0								(3338)
Mean flood for T=1.6→7.4 yrs		1649	2016	2369	2664	2900			2731
Error (%)			-39.6	-26.1	-13.3	- 2.5	+ 6.2		

The variable nature of the trend of required UCWI with return period makes the first two options somewhat impractical; and, therefore, option iii. has been adopted. The optimum UCWI for the catchment in question is obtained by taking the mean of each column in Table 5.1 for the four return periods between 1.6 and 7.4 years. The same procedure for the synthetic flood-frequency curve allows, by interpolation, an estimate of the most appropriate value of UCWI (in this case 82) and also an estimate of the error incurred in using a value of UCWI other than the optimum (or the sensitivity of the model to changes in UCWI). This analysis is shown at the bottom of Table 7, and is also included as the dashed line in Figure 5.2.

The procedure described above was performed for all ten chosen points in the Stevenage system. It was found that points near the bottom of the system gave a maximum peak discharge at the 30 minute duration, and those near the top at the 15 minute duration. An overall estimate of the best UCWI for the whole catchment is obtained by the method shown in Table 8. The last row of Table 7 has been entered as the first row of Table 8 for pipe 1.41. The results from the other points in the system are also shown, and the overall result taken as the mean of each column (one point - pipe 194.29 - had to be left out of this analysis due to the strange behaviour of the system in this region under circumstances involving heavy surcharging). From Table 8, it can be seen that the optimum UCWI for the Stevenage catchment is about 80. The last row in Table 8 indicates a measure of the model sensitivity to changes in UCWI. This last row is plotted in Figure 5.3, the absolute value of the error being plotted as the ordinate. The relationship plotted in Figure 5.3 can be compared to the results obtained from the other catchments, the analyses for which are described below.

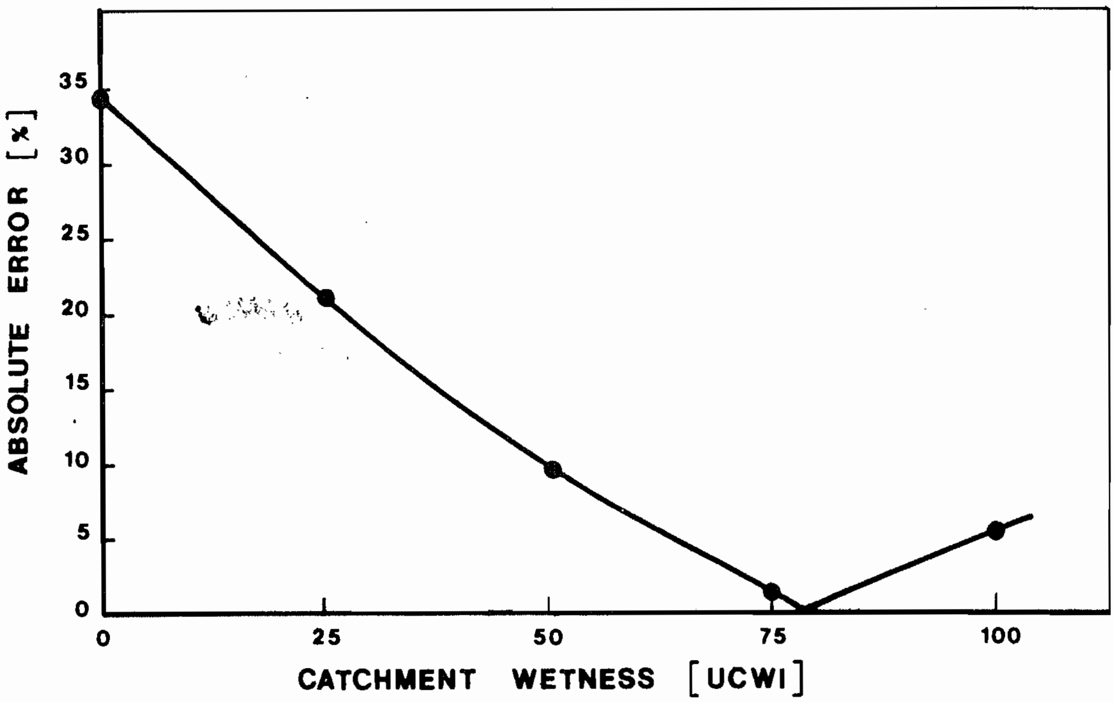

Figure 5.3 ABSOLUTE ERROR v. UCWI FOR STEVENAGE

TABLE 8: ESTIMATION OF BEST UCWI FOR STEVENAGE

Pipe No:	Mean error (%) for UCWI = 0	25	50	75	100
1.41	-39.6	-26.1	-13.3	-2.5	+6.2
1.37	-37.2	-23.6	-11.6	-1.9	+5.7
1.33	-34.5	-21.8	-10.1	-1.0	+5.4
1.14	-24.1	-12.7	- 2.8	-1.0	+3.4
113.30	-33.9	-20.1	- 9.3	-0.2	+6.2
113.23	-32.6	-19.3	- 9.0	-0.7	+5.6
113.14	-35.5	-20.1	- 7.3	+0.1	+5.3
113.12	-33.1	-19.9	- 9.6	-1.7	+5.7
31.14	-39.0	-26.3	-13.9	-3.4	+5.3
Mean	-34.4	-21.1	- 9.7	-1.4	+5.4

The analysis for Derby

The S.E. England rainfall series was again used for deriving the synthetic flood frequency curve for this catchment. As described in section 3, the runoff volume submodel was reduced to:

$$PRO = 37.0 + .078\ UCWI \qquad (5.2)$$

The standard error was 10%, and this value was used to generate a random noise component, as at Stevenage.

Figure 5.4 shows the flood-frequency curve obtained for pipe 1.23, together with an analysis of seven years of flow data. The fit at low return periods is not as good as that observed for the Stevenage catchment. However, subsequent scrutiny of the rainfall data for Derby (over the seven year period in question) showed that there were fewer observed rainfalls with return period ⩾ one year in the important durations (15 and 30 minutes) than might have been expected. For this reason, the synthetic flood frequency curve was considered to provide a fair estimate of the actual flood frequency distribution for the catchment. Table 9 and Figure 5.5 demonstrate the estimation of the best value of UCWI for pipe 1.26 (the bottom of the system). Table 10 brings together the results from all parts of the Derby system. Again, one pipe, 18.04, had to be excluded from the analysis due to its strange behaviour under conditions of heavy surcharging. The results show almost the same consistency in all parts of the system as was observed at Stevenage. There are two major differences however. Firstly, the optimum UCWI for the whole catchment is approximately 25 instead of the value of 80 from the Stevenage catchment. Secondly, the sensitivity of the model to changes in UCWI is far less than for the Stevenage catchment. This is to be expected due to the difference in the values of the SPR (37.0% for Derby and 7.78% for Stevenage) - since the runoff volume equation is additive, an incremental increase in UCWI will have a much smaller proportional effect on the percentage runoff (and thereby on the peak discharge) at Derby than at Stevenage. Because the Derby catchment is so insensitive to UCWI, the effect of adopting the Stevenage optimum (UCWI = 80) would only result in a 6% error at Derby. Adopting the Derby optimum (UCWI = 25) would result in a 21% error at Stevenage.

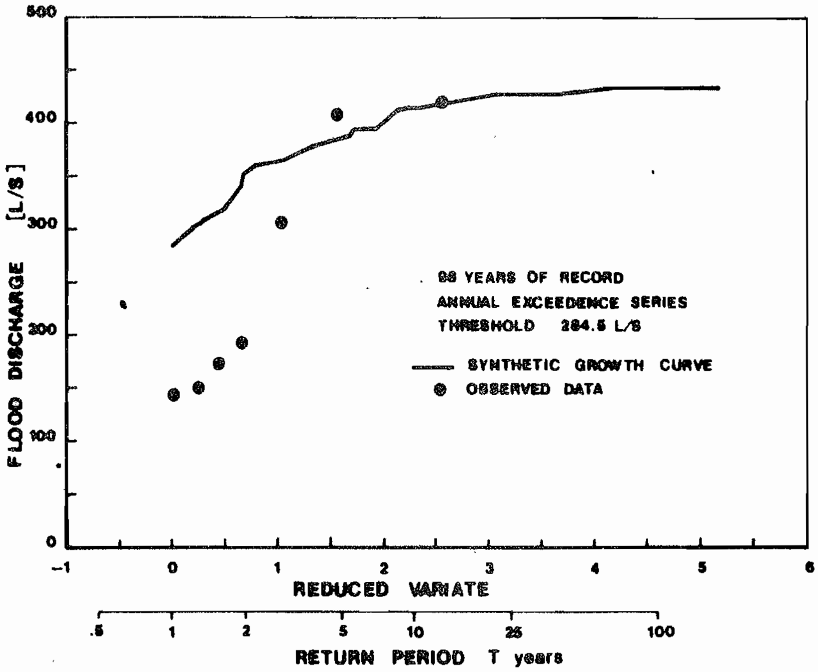

Figure 5.4 FLOOD FREQUENCY ANALYSIS FOR PIPE 1.23 AT DERBY

TABLE 9: TABLE OF FLOODS FOR PIPE 1.26 AT DERBY

Return period (yrs)	Plotting position	Catchment wetness (UCWI) 0	25	50	75	100	125	150	Synthetic growth curve
1.0	0	-	-	-	-	-	-	-	(353)
1.6	.5	396	441	439	440	447	-	-	409
2.7	1.0	429	444	454	461	497	-	-	448
4.5	1.5	468	485	505	521	524	-	-	489
7.4	2.0	505	527	541	542	560	-	-	528
12.2	2.5	-	-	-	-	-	-	-	(541)
20.1	3.0	-	-	-	-	-	-	-	(568)
Mean		452	467	485	491	507			468
Error (%)		-3.5	-0.4	+3.5	+4.8	+8.2			

TABLE 10: ESTIMATION OF BEST UCWI FOR DERBY

Pipe No:	Mean error for UCWI = 0	25	50	75	100
1.26	- 3.5	- 0.4	+ 3.5	+ 4.8	+ 8.2
1.23	- 2.2	+ 0.5	+ 3.1	+ 5.6	+ 7.2
1.17	- 1.9	- 0.1	+ 1.7	+ 3.5	+ 5.9
1.15	- 2.6	- 0.7	+ 1.1	+ 2.2	+ 4.3
1.07	+ 0.1	+ 3.8	+ 7.8	+ 8.6	+ 10.0
1.05	- 3.8	+ 2.1	+ 9.0	+ 10.0	+ 11.8
6.05	+ 2.3	+ 4.2	+ 6.3	+ 7.9	+ 8.7
17.04	- 12.0	- 9.3	- 6.1	- 1.9	+ 1.0
18.09	- 5.2	+ 1.3	+ 4.4	+ 7.3	+ 10.2
Mean	- 3.2	+ 0.2	+ 3.4	+ 5.3	+ 7.5

The analysis for ST1

As described in Section 3, ST1 is a hypothetical catchment designed to be a compromise between the Stevenage and Derby catchments. The S.E. England rainfall series was used to produce the synthetic flood-frequency curve. Similar results can be obtained for individual points in the system as have been shown for Stevenage and Derby, and only the overall results will be shown here. Table 11 shows the results of the sensitivity tests for the ten points in the system. The table shows that this catchment is a compromise between Stevenage and Derby in results as well as in design. The optimum UCWI is approximately 50, and the sensitivity of the model to changes in UCWI is greater than at Derby and less than at Stevenage.

TABLE 11: ESTIMATION OF BEST UCWI FOR ST1

Pipe No:	Mean error for UCWI = 0	25	50	75	100
1.41	- 13.1	- 4.9	+ 2.3	+ 8.6	+ 14.7
1.37	- 12.5	- 2.6	+ 4.3	+ 13.9	+ 19.9
1.33	- 12.9	- 4.2	+ 2.2	+ 10.5	+ 18.7
1.14	- 15.7	- 8.1	- 2.1	+ 4.0	+ 8.4
113.30	- 14.4	- 6.2	- 0.2	+ 6.9	+ 11.8
113.23	- 13.0	- 5.1	+ 1.0	+ 7.6	+ 11.8
113.14	- 12.5	- 6.1	+ 0.2	+ 6.1	+ 11.1
113.12	- 14.4	- 4.3	- 0.7	- 6.4	+ 13.2
31.14	- 16.4	- 9.5	- 1.5	+ 4.0	+ 9.6
194.29	- 15.1	- 10.1	- 1.6	+ 4.0	+ 12.2
Mean	- 14.0	- 6.4	+ 0.4	+ 7.2	+ 13.1

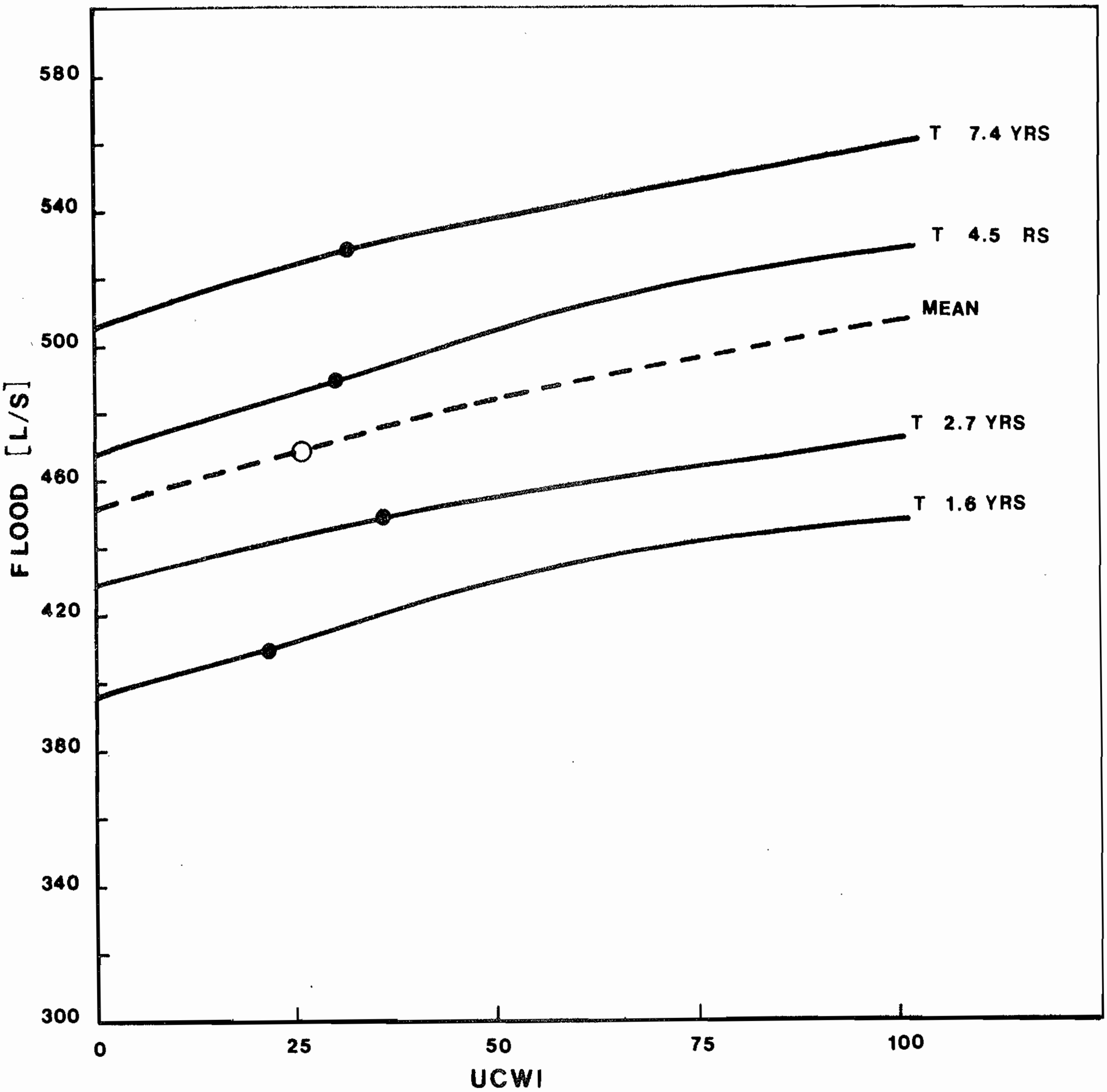

Figure 5.5 VARIATION OF FLOOD DISCHARGE WITH RETURN PERIOD AND UCWI FOR PIPE 1.26 AT DERBY

The analysis for ST2

ST2 is the same catchment as ST1 with some larger sizes at the top end of the system. It is assumed to be in a wetter part of the UK, and therefore the S.W. England rainfall series has been used in the derivation of the synthetic flood-frequency curve. As for ST1, only the overall results will be produced here. Table 12 shows the results of the sensitivity tests for the ten points in the system. These results show that the optimum UCWI for the ST2 catchment is approximately 100. This value is higher than had been obtained for the previous three catchments, which is to be expected due to the wetter rainfall series - section 4 demonstrated that the chief difference between the two series was the increase in average catchment wetness conditions. The model sensitivity to changes in UCWI is approximately the same as for ST1.

TABLE 12: ESTIMATION OF BEST UCWI FOR ST2

Pipe No:	Error at UCWI = 0	25	50	75	100	125	150
1.41	- 25.9	- 19.4	- 13.0	- 6.6	- 0.8	+ 4.7	+ 11.7
1.37	- 26.3	- 19.9	- 13.5	- 7.9	- 1.4	+ 5.1	+ 11.9
1.33	- 26.1	- 19.7	- 13.6	- 7.1	- 0.6	+ 6.8	+ 12.7
1.14	- 24.8	- 18.6	- 12.7	- 6.6	- 1.3	+ 3.6	+ 8.9
113.30	- 26.2	- 19.8	- 13.9	- 8.5	+ 0.1	+ 7.1	+ 12.1
113.23	- 23.8	- 17.2	- 10.9	- 4.9	+ 1.6	+ 6.6	+ 11.8
113.14	- 23.0	- 16.3	- 9.7	- 3.1	+ 4.4	+ 11.1	+ 17.6
113.12	- 21.3	- 15.0	- 9.0	- 2.1	+ 6.2	+ 13.7	+ 20.0
31.14	- 25.1	- 18.5	- 11.4	- 5.0	+ 0.4	+ 7.0	+ 12.1
194.29	- 28.8	- 22.6	- 16.6	- 9.6	- 1.1	+ 6.1	+ 12.8
Mean	- 25.1	- 18.7	- 12.4	- 6.1	+ 0.7	+ 7.2	+ 13.2

The analysis for Stevenage (SW)

This final catchment is the original Stevenage catchment in conjunction with the S.W. England rainfall series. The overall results of the sensitivity of the different points in the system (Table 13) show that the optimum UCWI for the Stevenage (SW) catchment is approximately 140, the sensitivity being of the same order as for the real Stevenage catchment.

TABLE 13: ESTIMATION OF BEST UCWI FOR ST2

Pipe No:	Error at UCWI = 50	75	100	125	150
1.41	- 28.9	- 18.7	- 10.6	- 3.2	+ 1.3
1.37	- 26.8	- 17.3	- 9.9	- 3.4	+ 0.7
1.33	- 23.9	- 14.3	- 8.3	- 3.4	+ 1.7
1.14	- 13.4	- 6.2	- 1.6	- 1.1	+ 2.7
113.30	- 22.7	- 15.0	- 8.2	- 2.9	+ 2.1
113.23	- 20.3	- 13.6	- 6.8	- 2.4	+ 2.8
113.14	- 16.6	- 10.1	- 3.5	- 1.8	+ 2.3
113.12	- 19.6	- 13.4	- 7.7	- 1.6	+ 2.7
31.14	- 28.1	- 18.5	- 12.3	- 6.1	+ 0.4
Mean	- 22.3	- 14.1	- 7.7	- 2.9	+ 1.9

Pooling the results

It is now possible to bring together the results from the five catchments to produce an overall picture. A plot of absolute error against UCWI (as in Figure 5.3 for Stevenage) has been produced for each catchment and put on a common base in Figure 5.6.

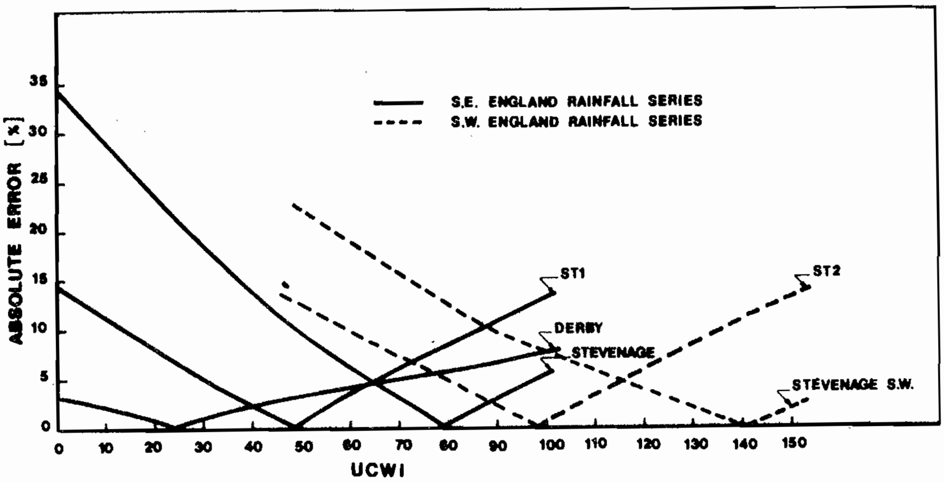

Figure 5.6 ABSOLUTE ERROR v. UCWI FOR ALL CATCHMENTS

Dealing with the S.E. England catchments first (the solid lines in Figure 5.6), it can be seen that the best overall result is obtained for UCWI = 65, at which point none of the three catchments is in error by more than 5%. If a 10% error is acceptable, then all catchments would lie within this threshold for 50 < UCWI < 85.

For the two S.W. England catchments, UCWI = 115 gives an error of less than 5% for both catchments. An error of less than 10% would be obtained for 90 < UCWI < 135.

In section 4, a relationship was establised between mean summer (June - September) UCWI and average annual rainfall (SAAR) for the UK. For convenience, this relationship is reproduced in Figure 5.7. Since arbitrary decisions have been made with respect to other design inputs (particularly storm profile), there is no *a priori* reason why the best UCWI values from the analyses described above should fall on this line - intuition would suggest a relationship of the form:

$$\{\text{mean (UCWI)} + \text{K.standard deviation (UCWI)}\} = \text{function (SAAR)} \quad (5.3)$$

would need to be produced. However, the two points plotted on Figure 5.7 for these analyses shows that no such adjustment is necessary, and that the design value of UCWI can be read from the mean (expected) summer UCWI v SAAR relationship. This result is very encouraging because it means that all the design inputs are now taking values which are realistic (in the sense that, profile apart, they all take values likely to occur in practice). It is therefore recommended that the value of SAAR for the catchment in question be used in conjunction with the relationship in Figure 5.7 to give a design value of UCWI - a knowledge of the accuracy of the techniques adopted suggests that the UCWI need only be estimated to the nearest 10.

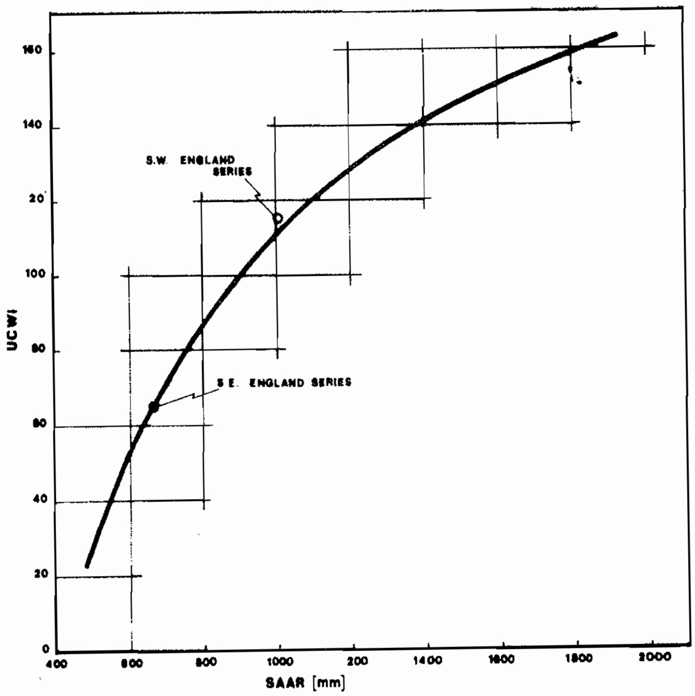

Figure 5.7 DESIGN CHOICE OF UCWI

The procedure gives a value of UCWI = 60 for the three S.E. England catchments (SAAR = 665) and a value of UCWI = 110 for the two S.W. England catchments (SAAR = 1002). These values have been used to generate design flood frequency distributions for all ten points in each catchment for comparison with their synthetic flood frequency distributions from the full simulation. A selection of these comparisons is shown in Figures 5.8 to 5.12. These show that as might be expected from the discussion earlier in the section the two flood frequency curves are well matched up to approximately a ten year return period. Above this figure, the design procedure progressively overestimates the synthetic flood frequency curve but the errors involved are not considered such that a change to a lower UCWI for rarer storms is necessary.

For comparison at Stevenage and Derby (Figures 5.8 and 5.9) the growth curve which would be obtained from the TRRL method (Watkins 1962) is also shown. The Stevenage outflow is overestimated by a factor of 1.8 at a one-year return period and by 2.5 at twenty-year return period. At Derby, the TRRL method gives a good estimate at low return periods but progressively overestimates at higher return periods due to the method's simplistic treatment of surcharging.

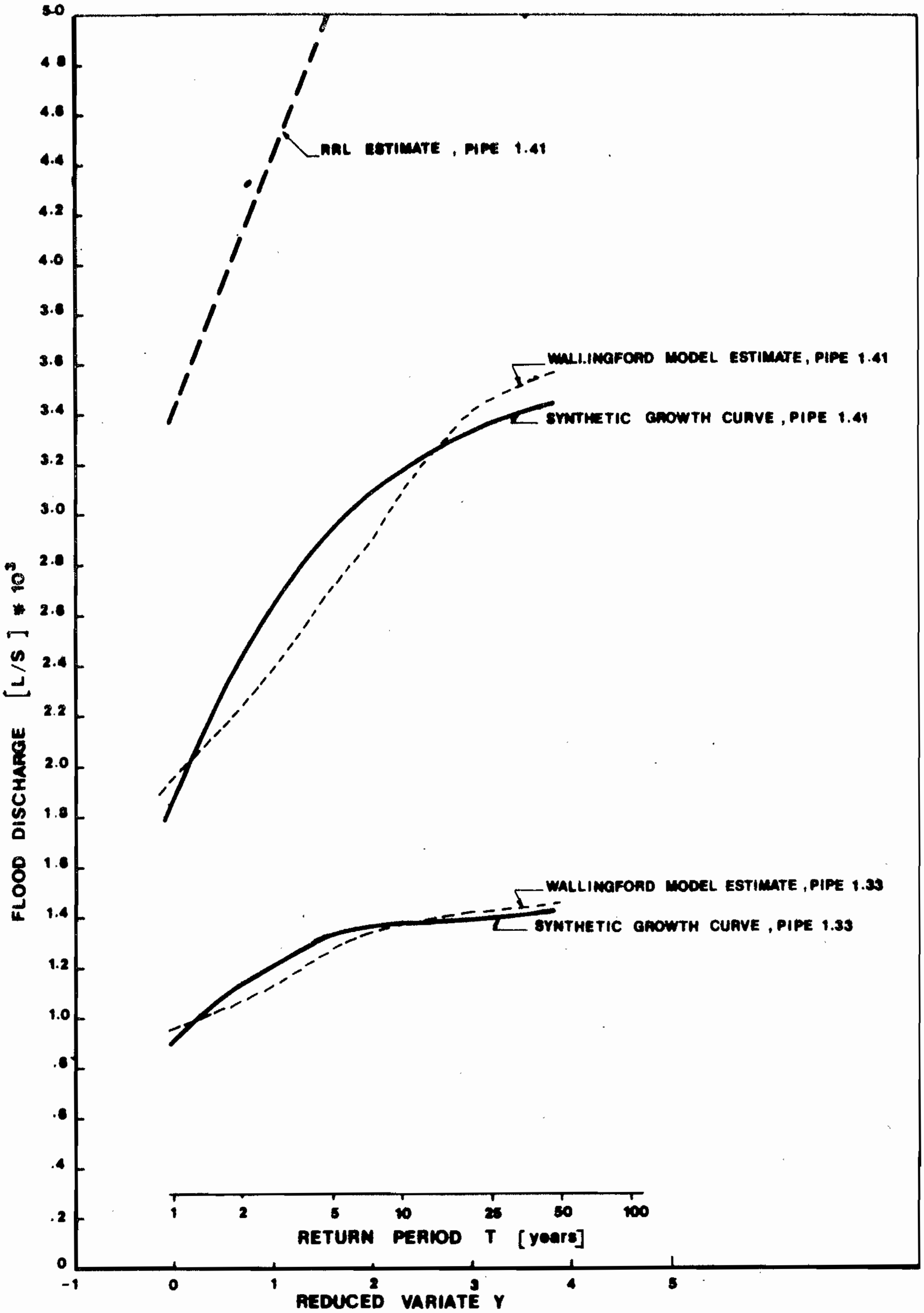

Figure 5.8 COMPARISON OF DESIGNED & SYNTHETIC FLOOD FREQUENCY DISTRIBUTIONS AT STEVENAGE

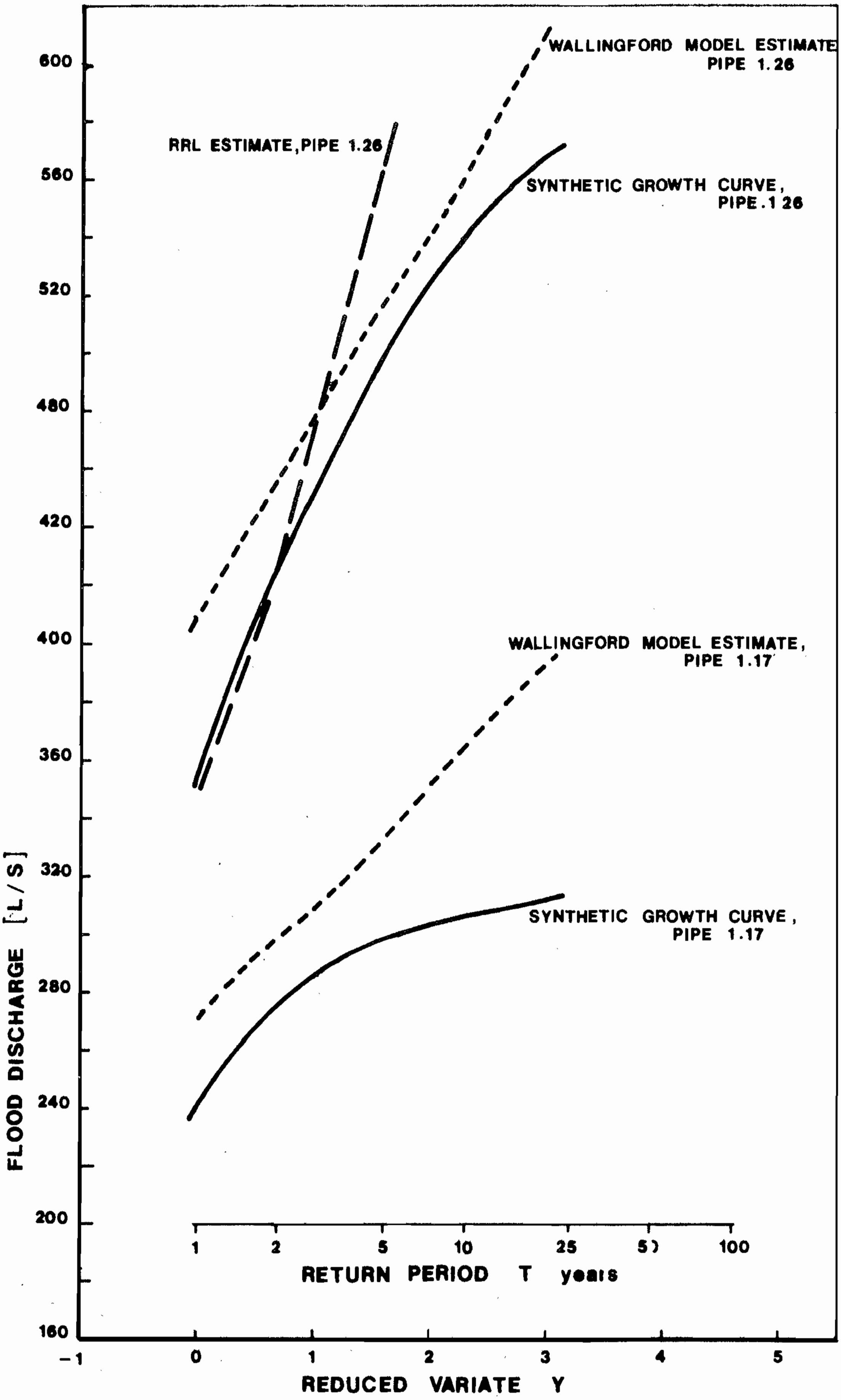

Figure 5.9 COMPARISON OF DESIGNED & SYNTHETIC FLOOD FREQUENCY DISTRIBUTIONS AT DERBY

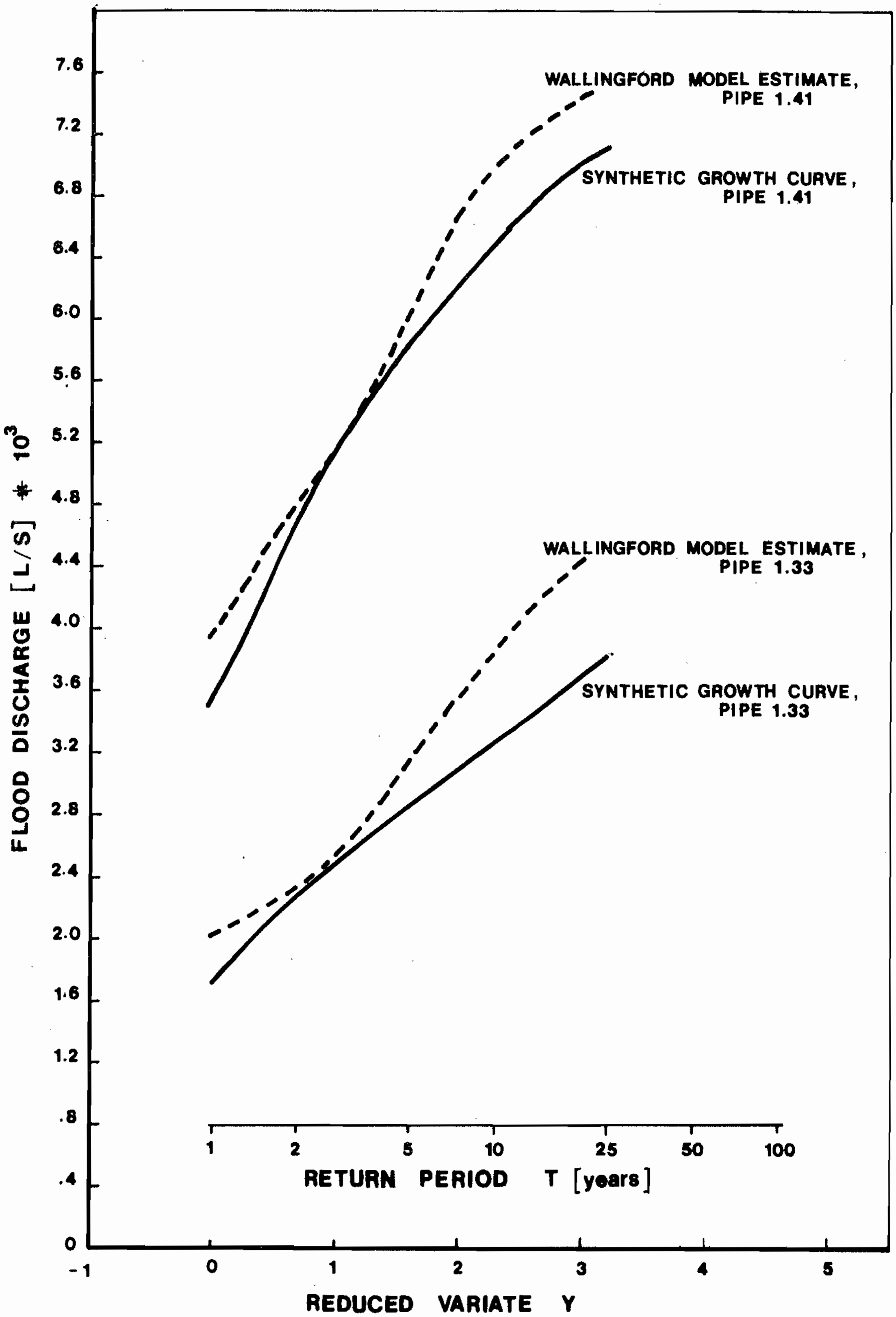

Figure 5.10 COMPARISON OF DESIGNED & SYNTHETIC FLOOD FREQUENCY DISTRIBUTIONS FOR ST1.

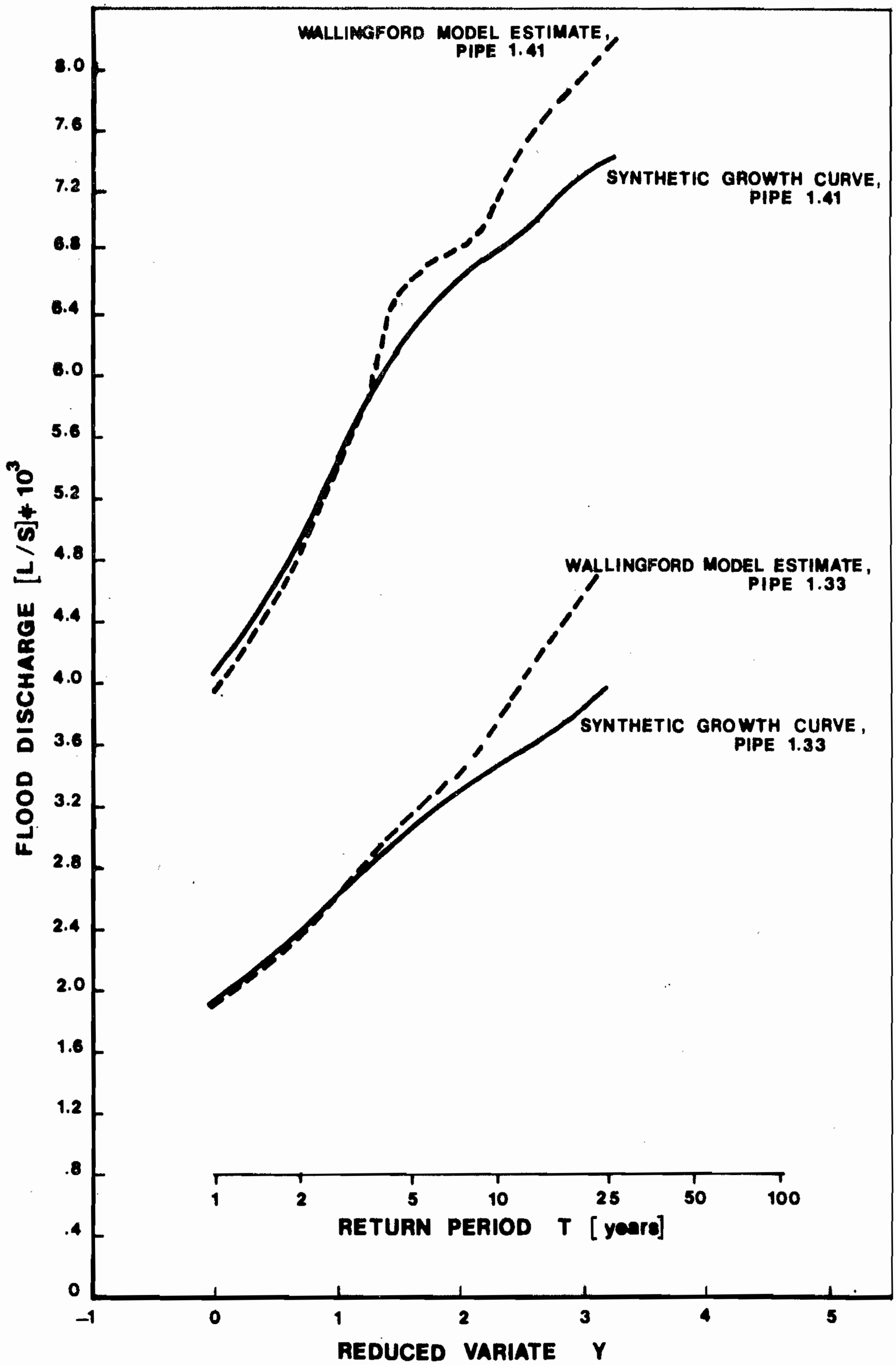

Figure 5.11 COMPARISON OF DESIGNED & SYNTHETIC FLOOD FREQUENCY DISTRIBUTIONS FOR ST2.

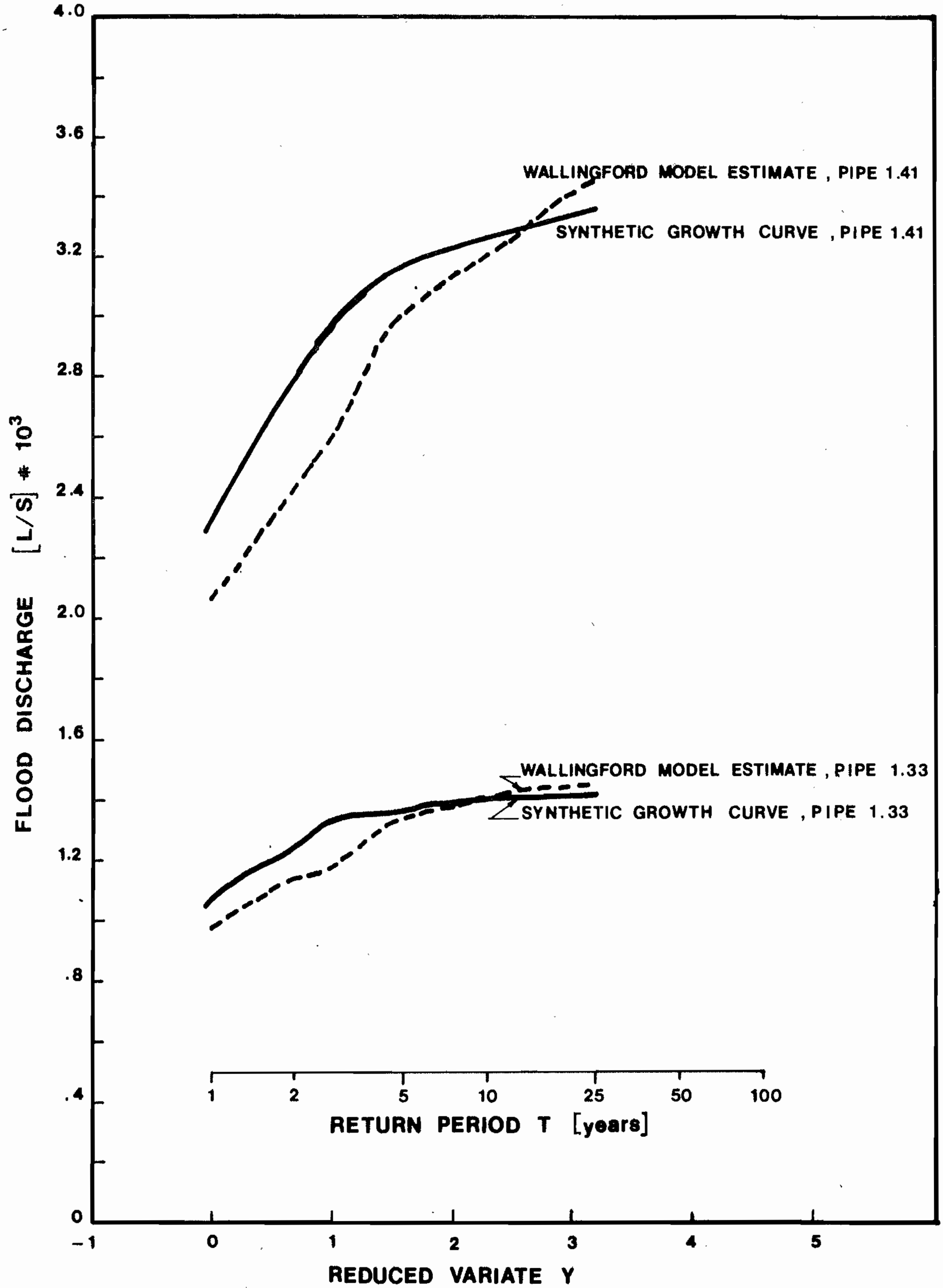

Figure 5.12 COMPARISON OF DESIGNED & SYNTHETIC FLOOD FREQUENCY DISTRIBUTIONS FOR STEVENAGE [S.W]

6. SENSITIVITY ANALYSES

The analyses undertaken in this study have allowed some general conclusions to be drawn on the sensitivity of the model to changes in the design inputs. These sensitivity tests are described in this section. The design inputs are:

i. rainfall return period (or total volume of rainfall)

ii. storm profile

iii. storm duration

iv. catchment wetness.

The model's sensitivity to changes in rainfall volume can also be presented as sensitivity to return period (thus avoiding the question of geographical location). Some examples are shown in Figure 6.1. In approximate terms, a 10% change in return period results in a 3% change in peak flow - a 10% change in rainfall volume (S.E. England) results in a 7% change in peak flow. The gradual flattening of the curves is presumably associated with surcharging - this phenomenon will also be observed for the other inputs.

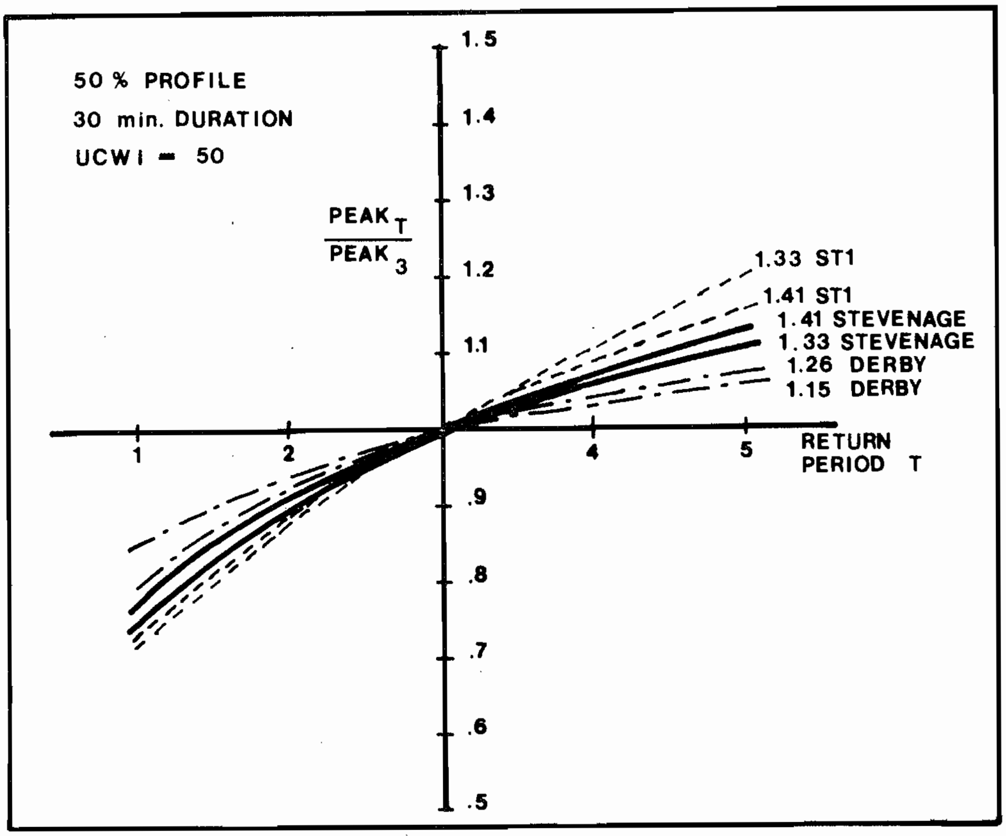

Figure 6.1 SENSITIVITY OF MODEL TO STORM RETURN PERIOD

The model's sensitivity to changes in storm profile is demonstrated in Figure 6.2. The particular profile can be seen simply as a convenient label and the 50% summer profile was the only one of the three considered (25%, 50%, 75%) which would allow reasonable values of the other inputs. Approximately a 10% change in profile is reflected in a 5% change in peak flow.

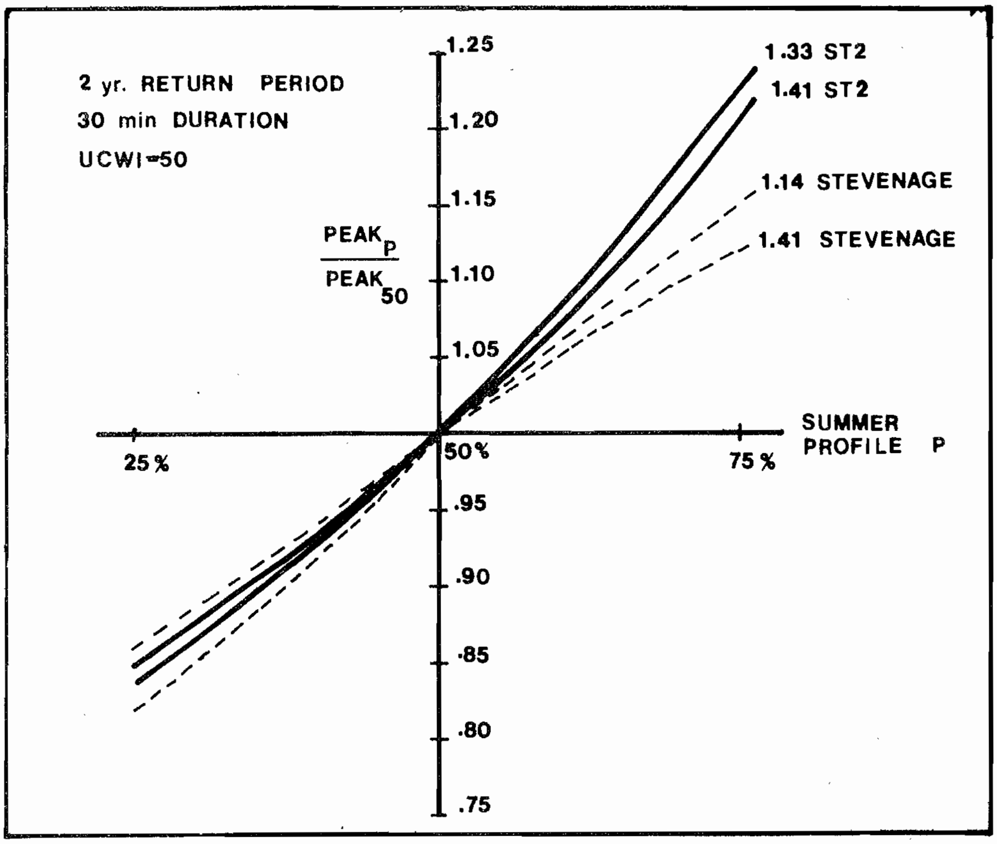

Figure 6.2 SENSITIVITY OF MODEL TO STORM PROFILE

The model is less sensitive to storm duration and Figure 6.3 demonstrates some results. These suggest that there would be little to gain from using a greater number of durations within the given range than the 15, 30, 60, 90 so far recommended. There is also little point in trying any of these durations if they are more than twice the time of concentration. In all our analyses, we never came across a critical duration greater than 30 minutes.

Figure 6.4 demonstrates the sensitivity of the model to changes in catchment wetness (UCWI). Since the runoff volume equation is additive we would expect the sensitivity of the model to changes in UCWI to depend on the SPR (ie that component of PRO associated with the other terms in the equation). Thus the model is much more sensitive to UCWI at Stevenage (10% changes in UCWI gives 3% change in peak) than at Derby (10% change in UCWI gives < 1% change in peak) where the SPR was much higher. The catchment named ST1 was designed as a compromise between Derby and Stevenage as is reflected in the model sensitivity in this respect. These conclusions are similar to those found in Section 5.

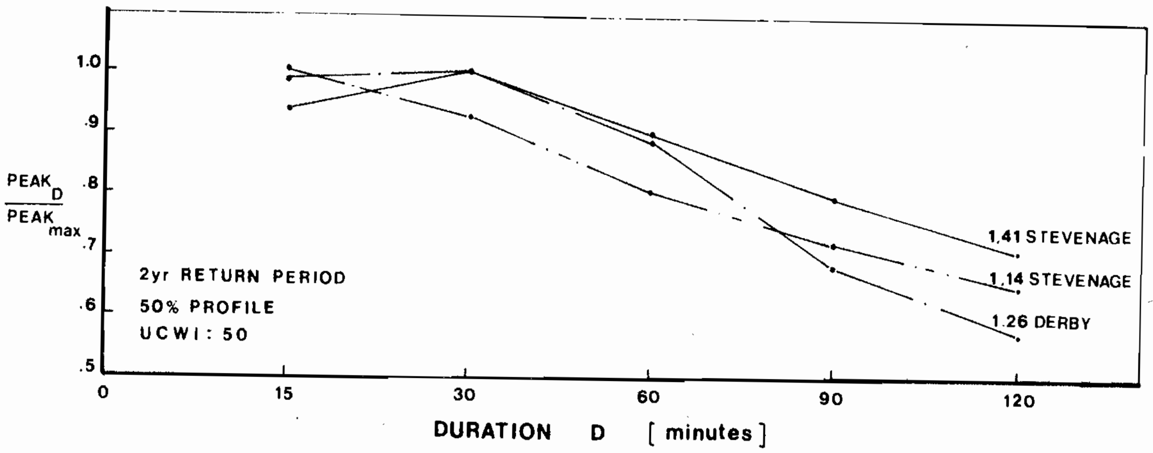

Figure 6.3 SENSITIVITY OF MODEL TO STORM DURATION

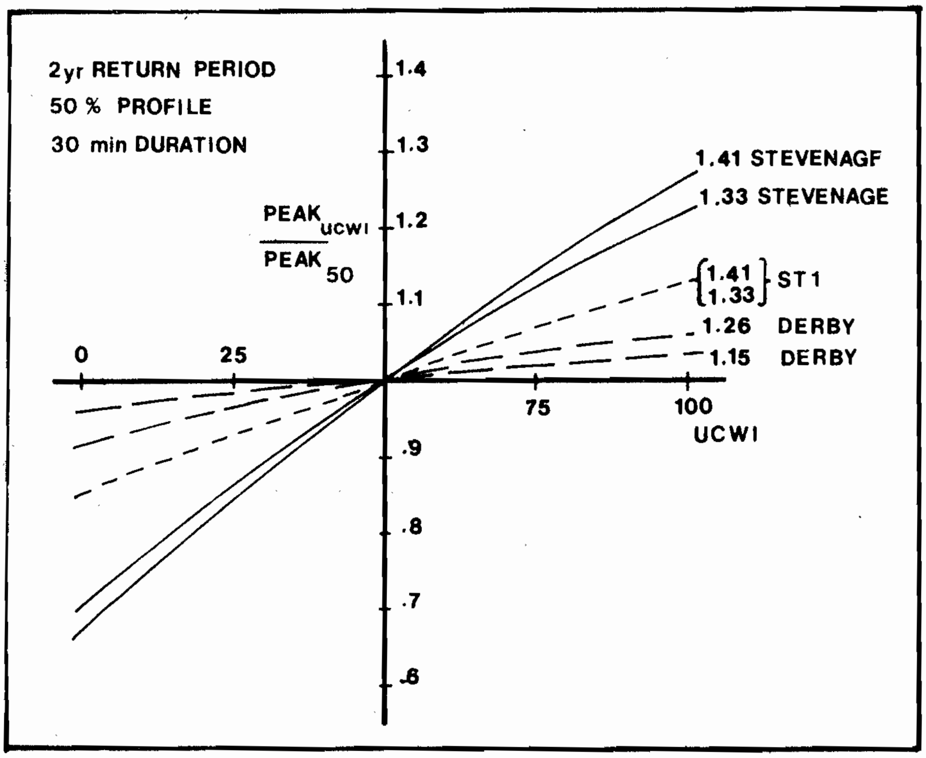

Figure 6.4 SENSITIVITY OF MODEL TO CATCHMENT WETNESS

7. AN ASSESSMENT OF THE RATIONAL FORMULA

Introduction

The data arising from the simulation analyses described in Section 5 have been used to assess the performance of the Rational Formula in design, with the possibility of producing a modified form based on these analyses.

The Rational Formula is given by:

$$Q = 2.78\ CIA \qquad (7.1)$$

where Q is the peak discharge (l/s),
I is the rainfall intensity (mm/hr) in some critical duration,
A is the catchment area (ha),
and C is a constant.

In the present form, the critical duration is taken as being equal to the time of concentration (time of travel + 5 minutes time of entry). Analyses may yield a different relationship for determination of the critical duration. Two versions of the model are now considered:

i. the Lloyd-Davies (1906) version where C is taken as equal to the proportion of paved surfaces within the catchment,

ii. an alternative version where C is the proportional runoff taken from the simplified model of the runoff volume:

$$C = .01 + .0074PIMP \qquad (7.2)$$

This equation derives from the same data set as was used to establish equation (2.2) and is described in more detail in a companion report (Kidd and Lowing 1979).

It remains to find the critical duration for any catchment for which equation (7.1) holds true. This analysis was undertaken for three of the catchments described earlier (Stevenage, Derby and ST2), and for a number of points within each catchment.

Stevenage analysis

Version 2 of the model can be assisted by a knowledge of the average percentage runoff (16.9%) of the events which contributed to the synthetic simulation. This suggests that the value of C at the bottom of the system (pipe 1.41) should be .169 instead of .196 as suggested by equation (7.2). For this reason, the other values of C for other points in the system have been adjusted pro rata.

Table 14 shows details of the analysis. Q_2 is the two-year return period discharge obtained from the synthetic simulation. I_2 is then the intensity derived from equation (7.1) and the critical duration is obtained from the depth-duration-frequency curve for the S.E. England

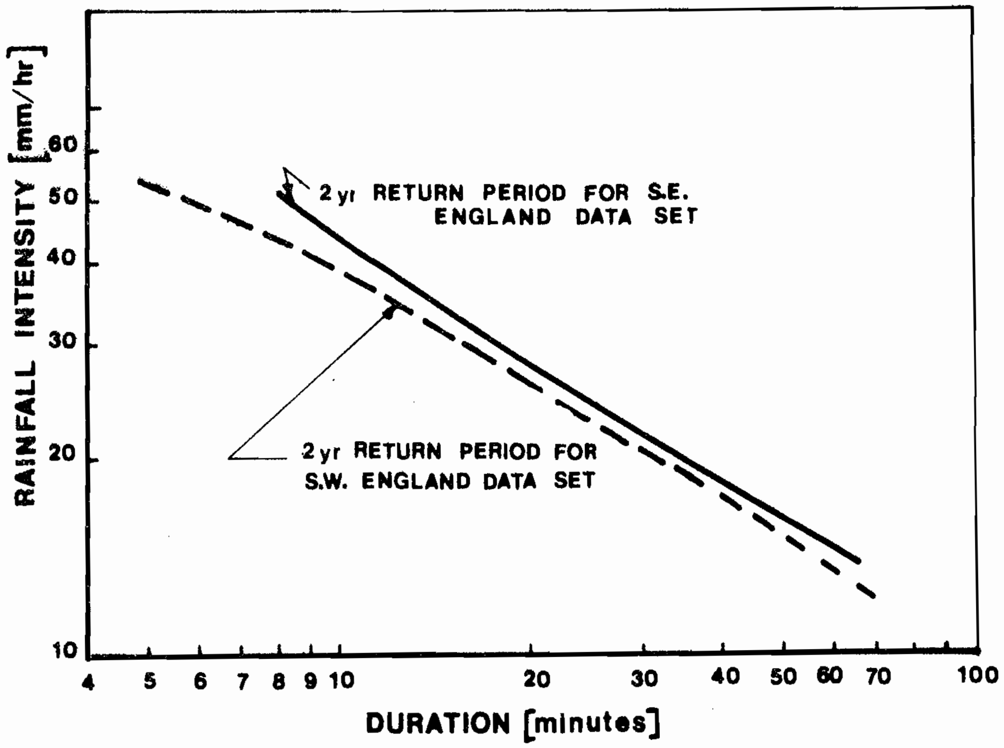

Figure 7.1 DEPTH DURATION FREQUENCY RELATIONSHIPS

rainfall series, which is shown in Figure 7.1. This curve is drawn through three points derived from Figure 2.3 in Section 2.

Derby analysis

The analysis was repeated for the Derby catchment, the only difference being that the corrected value of C at the bottom of the system (pipe 1.26) was in this case .461 instead of the value of .404 suggested by equation (7.2). Table 15 shows the results obtained.

ST2 analysis

ST2 is a hypothetical catchment sited in S.W. England. Its synthetic growth curve derives from a different rainfall set, and the corresponding depth-duration-frequency relationship is shown in Figure 7.1 (derived from Figure 2.5). The catchment is the same as Stevenage in layout, but the percentage impervious is higher and the pipes are at half the Stevenage slope. Table 16 shows the results of this analysis.

Analysis of the results

Figures 7.2 and 7.3 show a plot of the critical duration against time of concentration for versions 1 and 2 respectively. The two figures together demonstrate the scatter in results obtained due to the simplified form of the model. The safety factor afforded by the Lloyd-

TABLE 14: RATIONAL FORMULA ANALYSIS FOR STEVENAGE CATCHMENT

Pipe No.	Contrib. area (ha)	Percent imperv.	Q_2 (l/s)	Time of conc. (mins)	Version 1 C	Version 1 I_2 (mm/hr)	Version 1 Critical duration (mins)	Version 2 C	Version 2 I_2 (mm/hr)	Version 2 Critical duration (mins)	Q_2 estimate (Version 1) (l/s)	Error %
1.41	131.7	25.2	2420	18.0	.252	26.3	21.0	.169	39.1	12.9	2770	+ 14.3
1.37	102.1	27.1	2000	16.1	.271	26.0	21.6	.182	38.7	13.1	2460	+ 23.0
1.33	64.2	23.8	1110	15.6	.238	26.3	21.2	.160	38.9	13.0	1400	+ 25.8
1.14	23.0	18.2	319	11.7	.182	27.4	20.0	.125	39.9	12.6	465	+ 45.8
113.30	28.7	35.9	730	13.9	.359	25.5	22.1	.238	38.5	13.2	1007	+ 38.0
113.23	27.3	35.2	704	12.6	.352	26.4	20.9	.233	39.8	12.7	1014	+ 44.1
113.14	6.6	31.7	199	9.9	.317	34.2	15.1	.211	51.4	9.6	256	+ 28.5
113.12	3.7	26.2	94	9.2	.262	34.9	14.7	.176	52.0	9.4	125	+ 33.2
31.14	18.0	21.9	395	11.8	.219	36.1	13.9	.148	53.4	9.2	434	+ 9.8
194.29	26.4	17.1	480	11.9	.171	38.3	12.8	.118	55.5	9.0	493	+ 2.7

Mean = + 26.5%

TABLE 15: RATIONAL FORMULA ANALYSIS FOR DERBY CATCHMENT

					Version 1			Version 2				
Pipe No.	Contrib. area (ha)	Percent imperv.	Q_2 (l/s)	Time of conc. (mins)	C	I_2 (mm/hr)	Critical duration (mins)	C	I_2 (mm/hr)	Critical duration (mins)	Q_2 estimate (Version 1) (l/s)	Error %
1.26	10.3	53.2	434	23.4	.532	28.5	19.1	.461	32.9	15.9	426	- 1.8
1.23	8.5	51.3	355	21.9	.513	29.3	18.2	.445	33.8	15.5	339	- 4.8
1.17	7.2	48.9	279	19.1	.489	28.5	19.1	.424	32.9	15.9	282	+ 1.0
1.15	5.3	44.0	177	16.9	.440	27.3	20.1	.383	31.4	16.7	201	+ 13.8
1.07	2.7	44.8	92	10.4	.448	27.4	20.2	.390	21.4	16.7	143	+ 55.2
1.05	2.1	33.7	61	8.6	.337	31.0	17.1	.295	35.4	14.4	96	+ 57.9

Mean = + 20.3%

TABLE 16: RATIONAL FORMULA ANALYSIS FOR ST2 CATCHMENT

Pipe No.	Contrib. area (ha)	Percent imperv.	Q_2 (l/s)	Time of conc. (mins)	Version 1 C	Version 1 I_2 (mm/hr)	Version 1 Critical duration (mins)	Version 2 C	Version 2 I_2 (mm/hr)	Version 2 Critical duration (mins)	Q_2 estimate (Version 1) (l/s)	Error %
1.41	134.2	44.5	4680	18.3	.445	28.2	17.1	.332	37.8	10.5	4530	- 3.2
1.37	106.0	46.9	4010	16.3	.469	29.0	16.5	.350	38.9	10.0	4030	+ 0.6
1.33	64.4	42.6	2290	15.7	.426	30.0	15.6	.318	40.3	9.4	2290	0
1.14	21.8	34.6	880	11.8	.346	42.0	8.5	.261	55.7	4.7	735	- 16.4
113.30	32.4	57.0	1720	14.0	.570	33.5	12.8	.423	45.2	7.3	1640	- 4.6
113.23	30.6	56.3	1700	12.7	.563	35.5	11.6	.418	47.8	6.6	1630	- 4.3
113.14	7.1	52.6	402	10.1	.526	38.8	10.0	.391	52.1	5.0	400	- 0.4
113.12	3.8	45.8	196	9.5	.458	40.5	9.4	.342	54.3	4.5	193	- 1.3
31.14	17.7	40.0	772	11.9	.400	39.3	9.8	.300	52.3	5.0	692	- 10.3
194.29	24.6	32.9	730	12.1	.239	32.5	13.4	.248	43.0	8.0	782	+ 7.2

Mean = - 3.2%

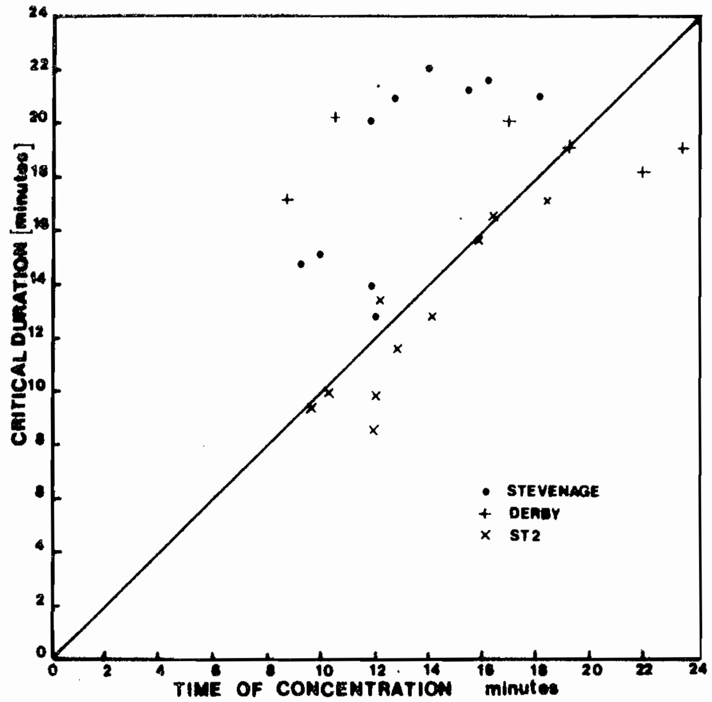

Figure 7.2 ANALYSES FOR VERSION 1 OF RATIONAL FORMULA

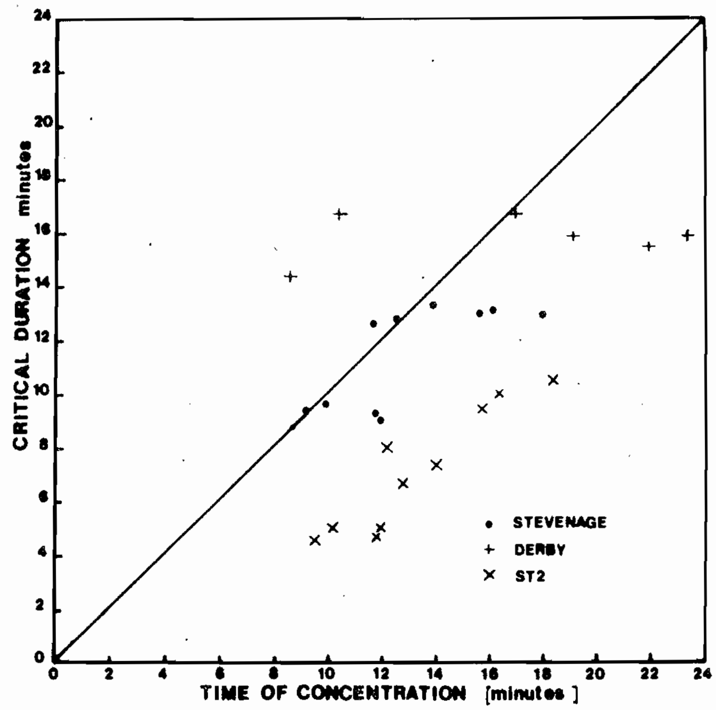

Figure 7.3 ANALYSES OF VERSION 2 OF RATIONAL FORMULA

Davies assumption (version 1) is demonstrated - this model in fact fits the ST2 catchment very well, while Derby and Stevenage will generally be overdesigned. A closer look at these results suggests that the Rational Formula as formulated begins to underdesign for times of concentration in excess of about 20 minutes. If evidence from three catchments is enough, Figure 7.3 suggests that severe underdesign could occur using the improved version 2.

From these results, it is felt that an alternative scheme for prediction of the critical duration cannot be obtained. The results ably demonstrate the variability in the accuracy of the prediction. It is proposed therefore that the original (version 1) form of the model should continue to be recommended, with the provision that for times of concentration greater than 20 minutes, a critical duration of 20 minutes should be used. Tables 14, 15 and 16 show the comparison between the two-year flood with the Rational Formula estimate as recommended. In this way, the Stevenage system would be overdesigned by an average of 26.5% (with variations between 2.7% and 45.8%); the Derby system would be overdesigned by an average of 20.3% (with variations between -4.5% and 57.9%); and the ST2 catchment is almost exactly right at an average of -3.2% (with variations between -16.4% and +7.2%).

It should be noted that these results are only relevant in circumstances where there is not extreme widespread surcharging. The same conclusion can be reached with other return periods until such time as the "observed" Q_2 derives from surcharged conditions - at this point the Rational Formula progressively overestimates the peak discharge.

8. CONCLUSIONS

An investigation has been undertaken into the relationship between the probabilities of hydrological variables in urban drainage design. This has allowed recommendations to be made as to the values of input variables to be used in conjunction with the Wallingford Model to obtain a peak flood of a required return period:

1. The design value of the catchment wetness UCWI can be obtained from the average annual rainfall (SAAR) according to the relationship

   $$\text{UCWI} = 233 - 1.51\times10^5/(\text{SAAR} + 237).$$

2. If a T-year rainfall volume with 50% summer profile over each of the durations of 15, 30, 60 and 90 minutes is applied to the Wallingford Model with the design value of UCWI (from 1. above), the maximum peak flow obtained from these durations will, on average, be the T-year flood for the catchment in question.

These statements have derived from analyses on a range of catchments and climatic conditions which will allow the Wallingford Model to be used in storm drainage design in the UK. Results from the above recommendations produced an error on the five catchments which was not in excess of 10% for return periods up to 10 years.

The Rational Formula has been examined in the light of the data produced in this study. This examination illustrated the scatter associated with these design estimates. On average, the Rational Formula will overdesign, although it may begin to underdesign for times of concentration in excess of 20 minutes.

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