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INSTITUTE of HYDROLOGY

FLOOD HYDROLOGY
OF THE RIVER NENE

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1.0 INTRODUCTION

The flood hydrology of the River Nene, between Northampton and Wansford, is complicated by the presence of many artificial controls on river flow (for navigation and flood protection) and by the significant flood plain storage. These factors render inappropriate the straightforward application of hydrological flood analysis techniques to individual sites along the river.

The Welland and Nene River Division of the Anglian Water Authority have judged that the most logical solution is to construct a mathematical model of flows in the River Nene and its appurtenant navigation and flood control structures. The effect of regime changes and the provision of extra storage areas or sluices on downstream river levels, when subjected to any hydrological input, can then be explored.

The broad aims of this study are to provide a set of hydrological inputs for gauged and ungauged tributaries, for both a design event (such that a reasonably constant return period is preserved at all points) and to assist in the calibration of the mathematical model. In addition, checks can be made that modelled river flows correspond, in their important statistics, to those observed in the historic record.

In order to achieve these aims, the specific objectives of this study are as follows:-

1. Derive unit hydrographs for gauged catchments and synthetic unit hydrographs for ungauged areas, in order to predict the response to any observed or design rainfall event.

2. Analyse past flood events, to build up a regional picture of the volumetric response of catchments.

3. Recommend a procedure for determining the design storm input and catchment response for a specified design event.

4. Flood frequency analyses for all gauging stations.

5. Provide AWA with a FORTAN subroutine to convolute rainfall and unit hydrograph data to yield a flood hydrograph.

The data which were available and useful to the study are described in Section 2. Section 3 outlines preliminary analyses and preparation of data. The methods of unit hydrograph and losses determination for the selected flood events are discussed in Section 4 and results presented for the gauged catchments. In Section 5 the ungauged areas are considered and their division into subcatchments and estimation of standard percentage runoff and timing of response is described. The procedure for calculating hydrological inputs for a calibration event is outlined in Section 6. In Section 7 the inputs for a design event, including specification of a suitable design storm are discussed. Frequency analyses of peak flows and volumes, which will aid calibration of the Nene model, are also presented.

The hydrologists at Welland and Nene River Division, namely P. Stott and N. Fawthrop, made a considerable contribution to the data collation phase of the study. N Fawthrop obtained and collated all autographic rainfall and flow charts, assisted in the selection of events, and extracted annual peak discharges from flow records, for use in the flood frequency study. They also provided much valuable advice on the quality of data, including observations on catchment response from past flood events, which could be incorporated into the analysis.

2.0 HYDROLOGICAL AND METEOROLOGICAL DATA AVAILABLE FOR THE STUDY

A plan of the Nene Catchment, upstream of Wansford is shown in Figure 1. The mathematical model of the River Nene has been developed for nontidal reaches, so Wansford is the downstream point of applicability. The location of hydrometric sites within this area is indicated in Figure 2 and Table 1. The Nene itself is gauged downstream of Northampton, but the stage - discharge relationships are unstable and influenced by sluice gate openings and other artificial controls. These sites are therefore unsuitable for rainfall runoff analyses.

As indicated in Figure 2 there are five main gauged tributaries which together comprise 53% of the total catchment area upstream of Wansford. Most of these tributaries are gauged at several sites, but to reduce channel routing to a minimum only the most downstream station on each tributary was utilised in the analyses. Other tributaries are gauged in the low and medium flow range only and although unsuitable for flood analyses provide useful additional information. Level hydrographs give an indication of the speed of catchment response and low flow stations provide infilling information on base flows to which flood response can be related.

2.1 Flow data

Approximately 10 events were selected from river level charts for each catchment from the period 1973 to 1979, these are listed in Table 2. Events prior to 1973 were not considered since autographic rainfall data were very sparse before that date. It was hoped to make use of the same rainfall event at many or all of the stations.

Events were selected using the following criteria.

(i) The magnitude of the flood peak (to include as many high flow events as possible).

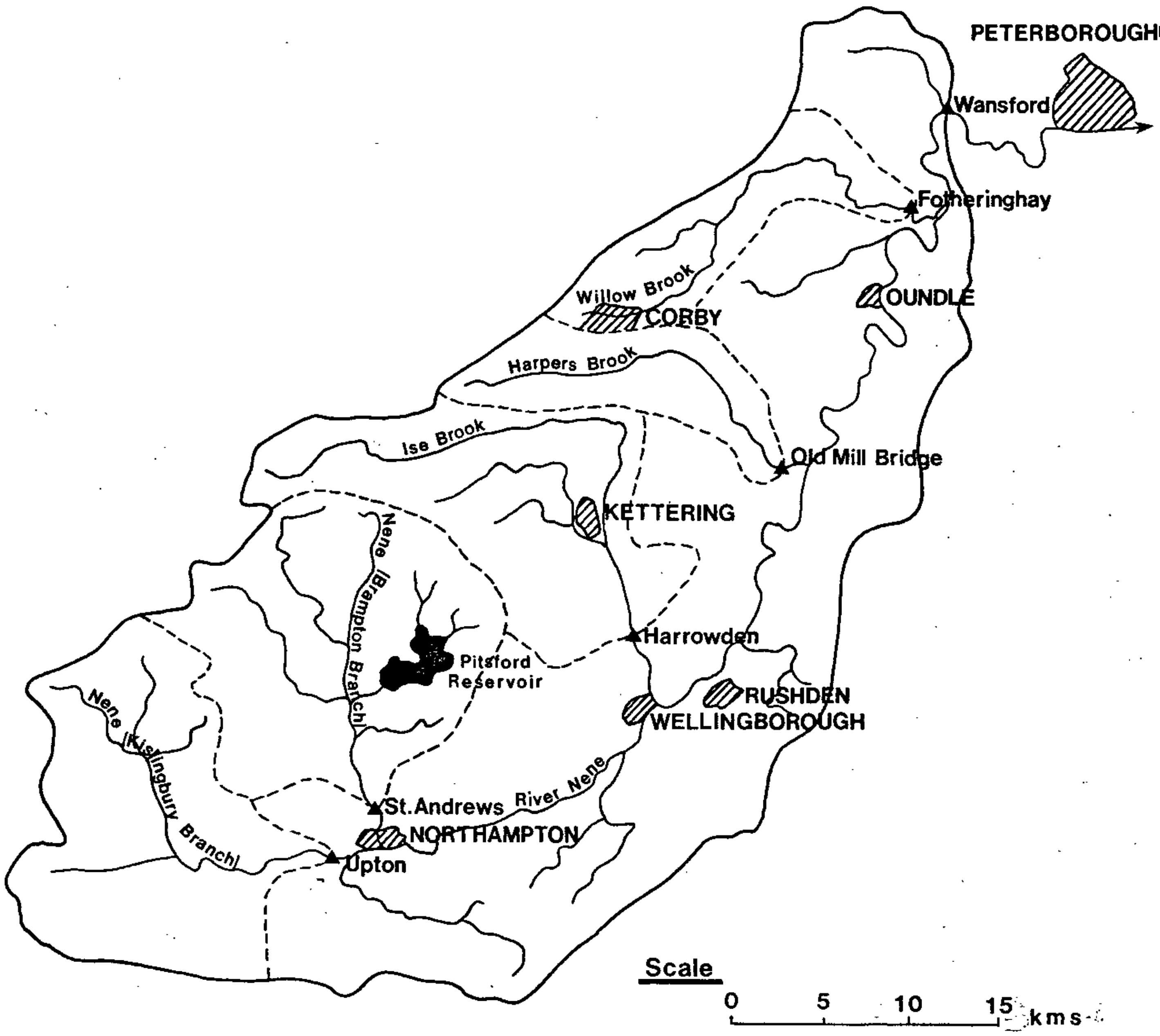
(ii) Events with single peaked hydrographs, where possible.

(iii) Include both summer and winter events

(iv) Flood peaks should be independent of each other.

(v) Exclude snowmelt events.

Daily flow data was required for the base flow analysis and volume flood investigation of Section 7.2. These were obtained from the Water Data Unit and from the River Division, as appropriate.

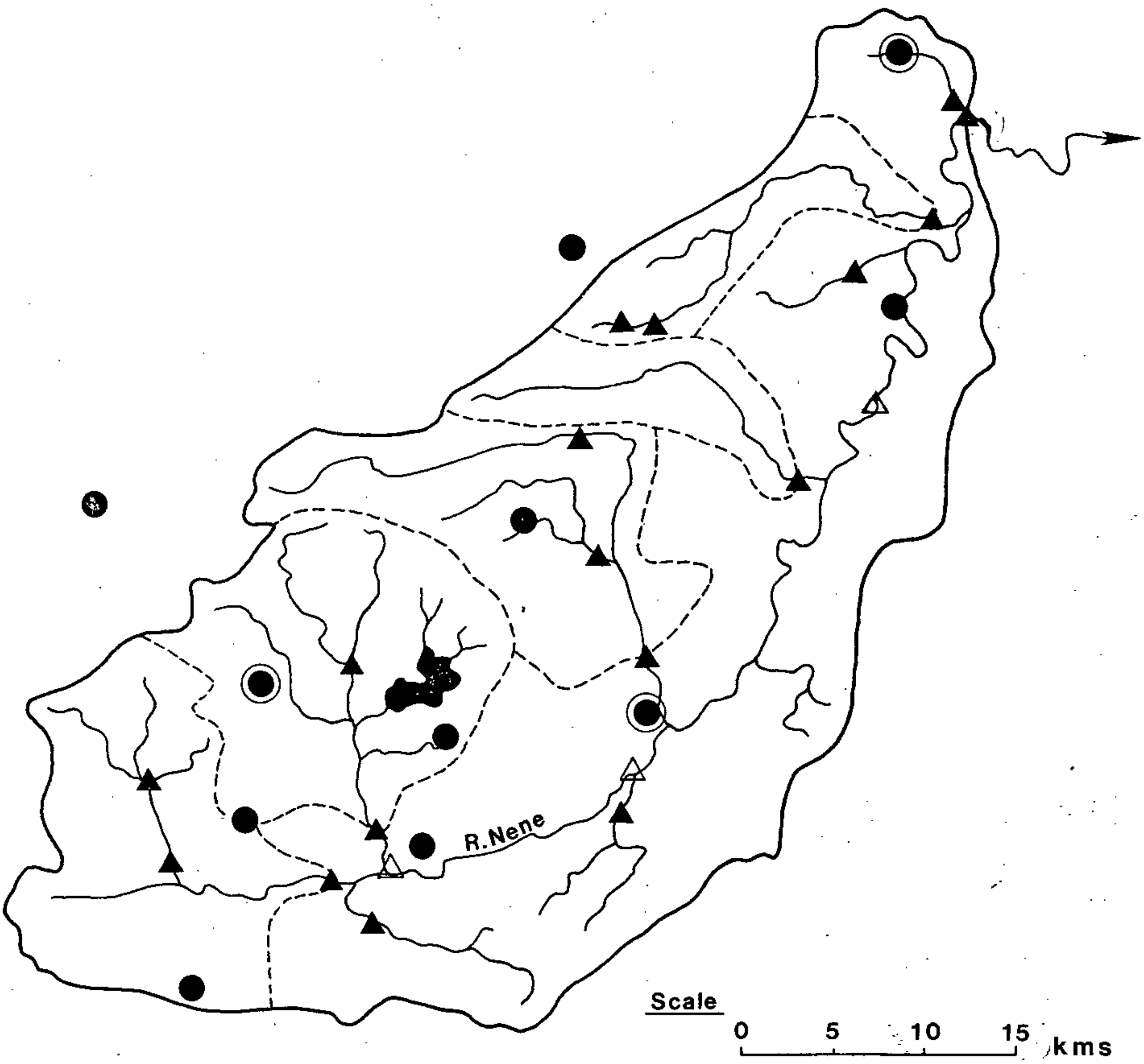


▲ gauging station

gauged tributaries used for hydrograph analyses

urban area

Figure 1. River Nene - catchment plan



▲ river flow gauging station

△ river level gauging station

● autographic raingauge

○ smd station

Figure 2. Hydrometric sites in River Nene catchment

Table 1 Full Range and Low Flow Gauging Stations in Nene Catchment.

Station Number	Station and River Names	Catchment Area Km^2	
32002	Willow Brook at Fotheringhay	89.62	
32003	Harpers Brook at Old Mill Bridge	74.59	
32004	Ise Brook at Harrowden Old Mill	194.00	
32005	Nene at Northampton	569.80	Levels
32006	Nene (Kislingbury Branch) at Upton	223.00	
32007	Nene (Brampton Branch) at St Andrews Mill	232.80	
32008	Nene (Kislingbury Branch) at Dodford	107.00	
32009	Willow Brook at Blatherwyke Lake	.00	
32010	Nene at Wansford	1528.30	
32012	Wooton Brook at Ladybridge	53.00	
32013	Nene at Wollaston	645.00	Levels
32014	Nene at Lilford	1258.00	Levels
32015	Willow Bk (Central St) at Tunwell Loop	7.10	
32016	Willow Bk (South St) at Stanion Lane	7.60	
32018	Ise at Barford Bridge	62.40	
32019	Slade Brook at Kettering	58.30	
32020	Wittering Brook at Wansford	46.90	LF
32023	Grendon Brook at Ryholmes Bridge	47.50	LF
32024	Southwick Brook at Southwick	20.50	LF
32025	Nene at Surney Bridge	63.40	Levels
32026	Nene (Brampton Branch) at Brixworth	58.00	
32801	Flore Experimental Catchment	7.00	

LF = Low flow station

Levels = 'Level only' station i.e. no rating curve.

Table 2 Event Selection

Date	Nene at Upton (32006)	Nene at St Andrews (32007)	Ise Brook (32004)	Harpers Brook (32003)	Willow Brook (32002)
27.6.73	*	*	*	*	*
17.11.74	*	*	*	*	*
19.1.75	*	*	*	*	
8.3.75	*	*	*	*	
18.4.75	*	*	*	*	*
30.12.76	*	*	*	*	*
27.1.77				*	*
23.1.78	*				
26.1.78	*				
5.5.78	*		*	*	*
23.12.78	*	*		*	*
7.4.79			*	*	*
26.5.79	*	*			*
Total	11	8	8	10	9

2.2 Rainfall data

The recording rainfall data, used in the study, took several forms: (i) charts, which were digitised on the IH D-mac digitiser and hourly rainfall totals extracted by computer program (ii) hourly tabulations, already analysed from the Meteorological Office (iii) hourly tabulations, analysed by computer from the punch tape recorders on Welland and Nene's tipping bucket gauges. The locations of the autographic gauges are shown in Figure 2.

For events prior to 1978 daily rainfall data were extracted automatically from the Meteorological Office's "British Rainfall" magnetic tapes using a computer program. For post January 1978 events, daily rainfall was extracted manually from computer listings, as British rainfall tapes were unavailable.

2.3 Soil moisture deficit data

Soil moisture deficit data were obtained from the Meteorological Office for the starting day of each event, for the SMD stations nearest to the catchment. There are three SMD stations in the Nene catchment, located as shown in Figure 2 and where applicable the average of two stations was taken.

3.0 DATA PREPARATION AND ANALYSIS

3.1 Flow data

The river level charts for the events selected in Section 2.1 were digitised using a D-mac pencil follower, which outputs the stage hydrographs in digital form suitable for computer analysis. A computer program then converts stage to discharge, hourly values are interpolated and hydrographs plotted using a Calcomp plotting program. Stage-discharge relationships were in all cases provided by Welland and Nene River Division.

At the Nene at Upton (32006) and St Andrews (32007) gauging stations, part of the flow is diverted through a bypass channel, where it is gauged separately. Both the channel and by-pass charts were therefore digitised and analysed, and the flows summed using a computer program, to form the total catchment outflow. Because of occasional timing differences in the chart pairs, this program checked that the peaks of the individual hydrographs occurred in the same time increment.

The stage hydrographs for the Ise Brook (32004), Nene at Upton and St Andrews catchments all exhibited distinct features which could be attributed to artificial influences. For example, many of the hydrographs at St Andrews were preceded by a sharp rise then fall in level, reflecting the initial rapid response of the urban area of Northampton, which is immediately upstream of the gauging station. Althoughthese 'blips' are significant in terms of the peak discharge and total catchment runoff, they induce instability into the numerical derivation of the unit hydrograph, and the stage hydrograph was smoothed prior to digitization.

At Ise Brook, Nene at Upton, and Willow Brook (32002) catchments the operation of gates, and other artificial controls on river level, results in sudden falls in level followed by a more gradual rise. These are most visible on the recession limbs of the flood hydrograph, and were smoothed out prior to the analysis (with care taken to preserve volumes).

3.2 Rainfall data

A catchment average rainfall profile for each event was derived using the method outlined in the Flood Studies Report (Vol 4, 3.2). An isopercental method is used to compute total storm rainfall, using all available daily gauges both on and in the vicinity of the catchment, as defined by an enclosing quadrilateral. This storm total is distributed in time according to the hourly pattern of any autographic gauges located within this quadrilateral, weighted according to their distance from the centre of the catchment. A one hour time interval was utilised in all analyses, and the timing of all flow and rainfall charts reduced to Greenwich Mean Time, where this was not already the case.

Alternative methods for deriving catchment average rainfall include the isohyetal and Thiessen polygon methods. The isohyetal method is generally inappropriate for automatic data handling and the Thiessen method involves lengthier computations. In an area of relatively uniform topography such as this, with a large number and fairly even distribution of daily gauges, the application of the Thiessen method was felt to be unwarranted. The isopercental method, outlined above, seemed to provide the best compromise between accuracy and speed of computation. In addition, since the average is derived from the daily fall at each gauge, expressed as a percentage of the standard average rainfall of that gauge, it implicitly allows for any systematic variation due to topography.

The computer program also extracts, for the gauge nearest to the centre of the catchment, the daily rainfall for the five days prior to the start of an event. This enables calculation of the five day antecedent precipitation index (API5), as outlined in the Flood Studies Report (Vol.I, 6.4.4).

3.3 Flood frequency data

All five Nene catchments have been gauged since between 1939 to 1945. As there are over 30 years of records at all sites, the flood frequency study was based on an annual maximum series. Instantaneous annual maxima prior to 1973 were available in the Floods Studies Report and were extracted directly from flow records for 1973 onwards by Welland and Nene River Division.

At the Nene at Upton catchment (32006) it was noticeable that peak flows after 1968 were consistently higher than those in the previous 25 years. This discrepancy may be due in part to channel improvements, resulting in increased capacity or inadequacies in the method of calculating flows in the bypass prior to 1968. Whatever the cause, the data series is clearly inhomogeneous and pre 1968 maxima were excluded from any frequency analyses. Annual maxima were also extracted for the Nene at Wansford (32010), for use in the formulation of a design event.

3.4 Comparisons of catchment response

The responses of the five gauged tributaries to the same rainfall inputs were compared to help elucidate any trends or consistencies in both the timing and nature of the response, which would be useful in modelling.

Flow hydrographs for the five tributaries and the Nene at Wansford, resulting from two storms on 18/4/75 and 30/12/76 are shown in Figures 3 and 4. Catchment average hyetographs for the Nene at Upton (32006) and Willow Brook (32002) catchments are also plotted, as an indication of the spatial uniformity of the rainfall. There was no evidence of storm movement for either events, but rainfall intensities increased eastwards during the April 1975 storm. A proportionately smaller response might be expected from the headwater catchments for this event, than if rainfall had been spatially uniform.

There was a fairly consistent response to both storms with hydrographs responding 1 - 2 hours after the commencement of rainfall. Harpers Brook (32003) showed the flashiest response, followed by Upton 1 hour later and the St Andrews peak 5 - 6 hours after that. Both Willow Brook and Ise Brook start to peak at approximately the same time as Harpers Brook but the peak is sustained (with irregularities) for 15 - 20 hours.

The Wansford hydrograph is more attenuated, but shows the same rapid rise after rainfall as a result of runoff from Willow Brook and other adjacent tributaries. The hydrograph peak is typically sustained for several days reflecting the effects of translation of distributed inflows along the Nene. Figures 3 and 4 suggest that lag times from Upton to Wansford are typically about 48 hours. Examination of these and other observed Wansford hydrographs have indicated that there is a sustained peak which should be reflected in any design hydrograph.

Figure 3. Comparisons of catchment response to storm on 18/4/75

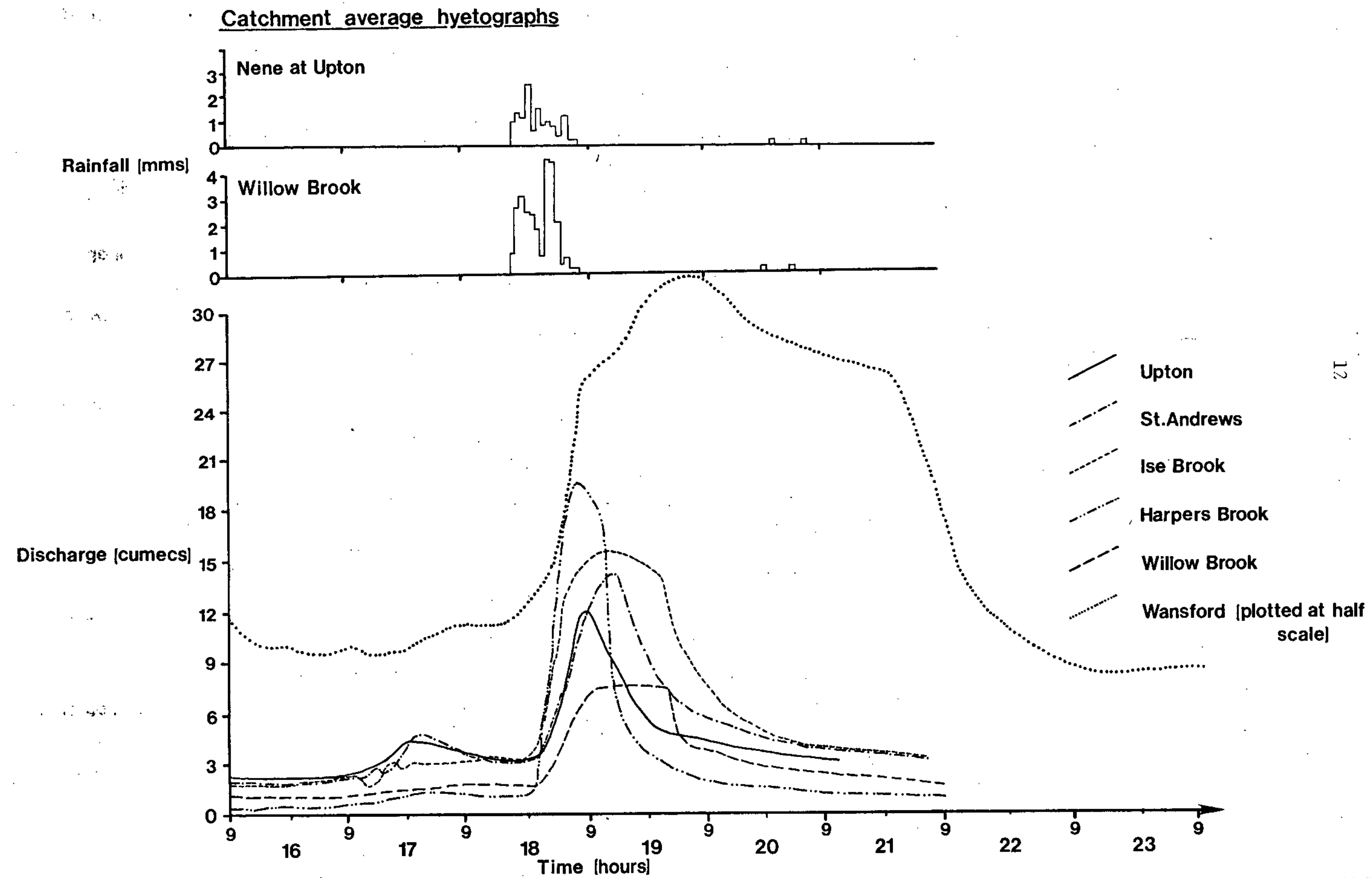
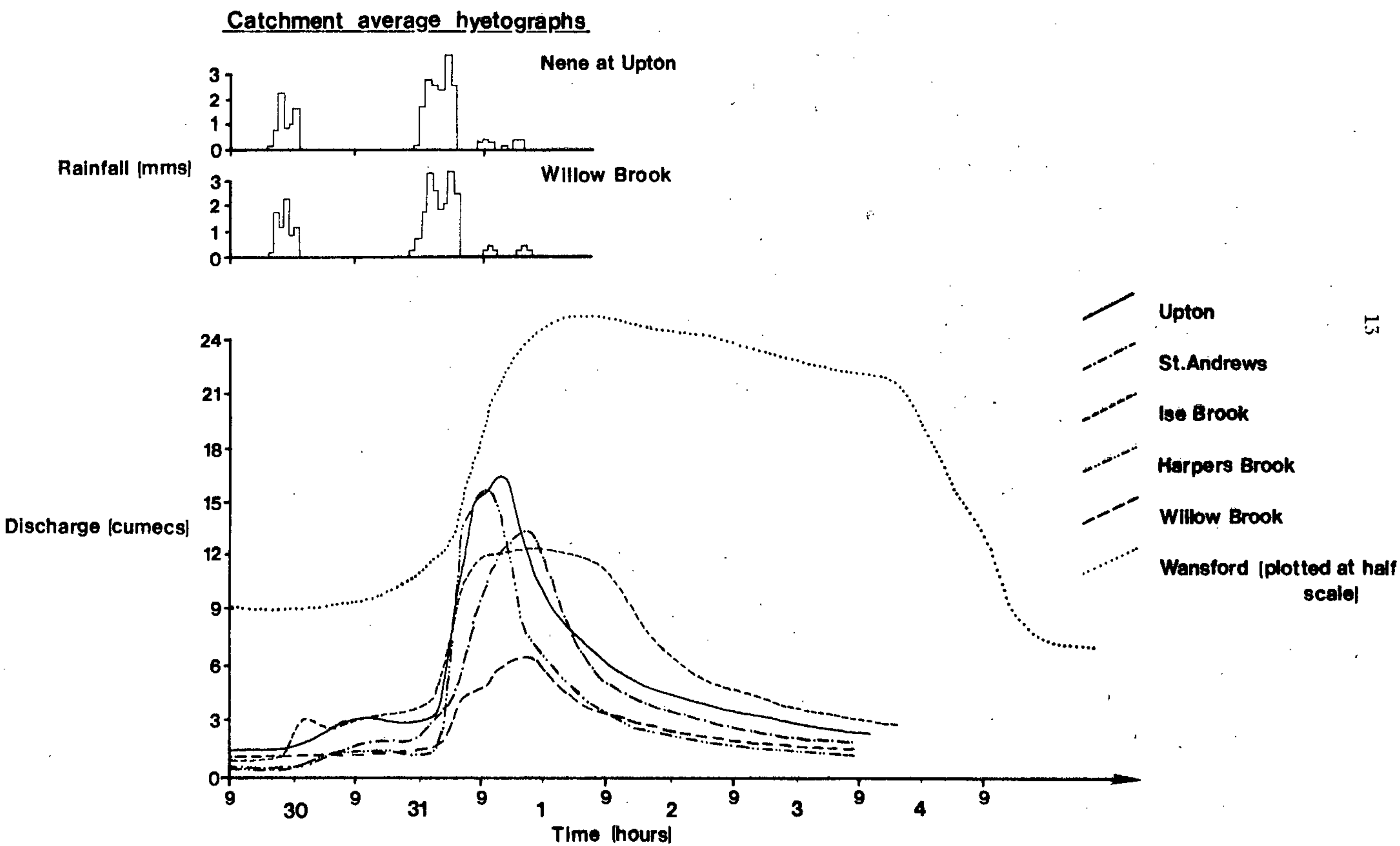


Figure 4. Comparisons of catchment response to storm on 30/12/76



4.0 FLOOD HYDROGRAPH ANALYSIS TECHNIQUES

4.1 Losses Study

The four types of input data - flow, rainfall, SMD, API5 were assembled on the computer and used in the following calculations:-

(i) Catchment lag - this is defined as the time from the centroid of the total rainfall hyetograph to the time of peak flow. In the case of multiple peaked hydrographs the centroid of these peaks is used.

(ii) The baseflow component is subtracted from the total flow hydrograph to yield a quick response hydrograph.

(iii) Catchment wetness index (Vol 1, 6.4.4.) - to indicate the state of the catchment prior to the start of an event. This index, CWI is calculated as

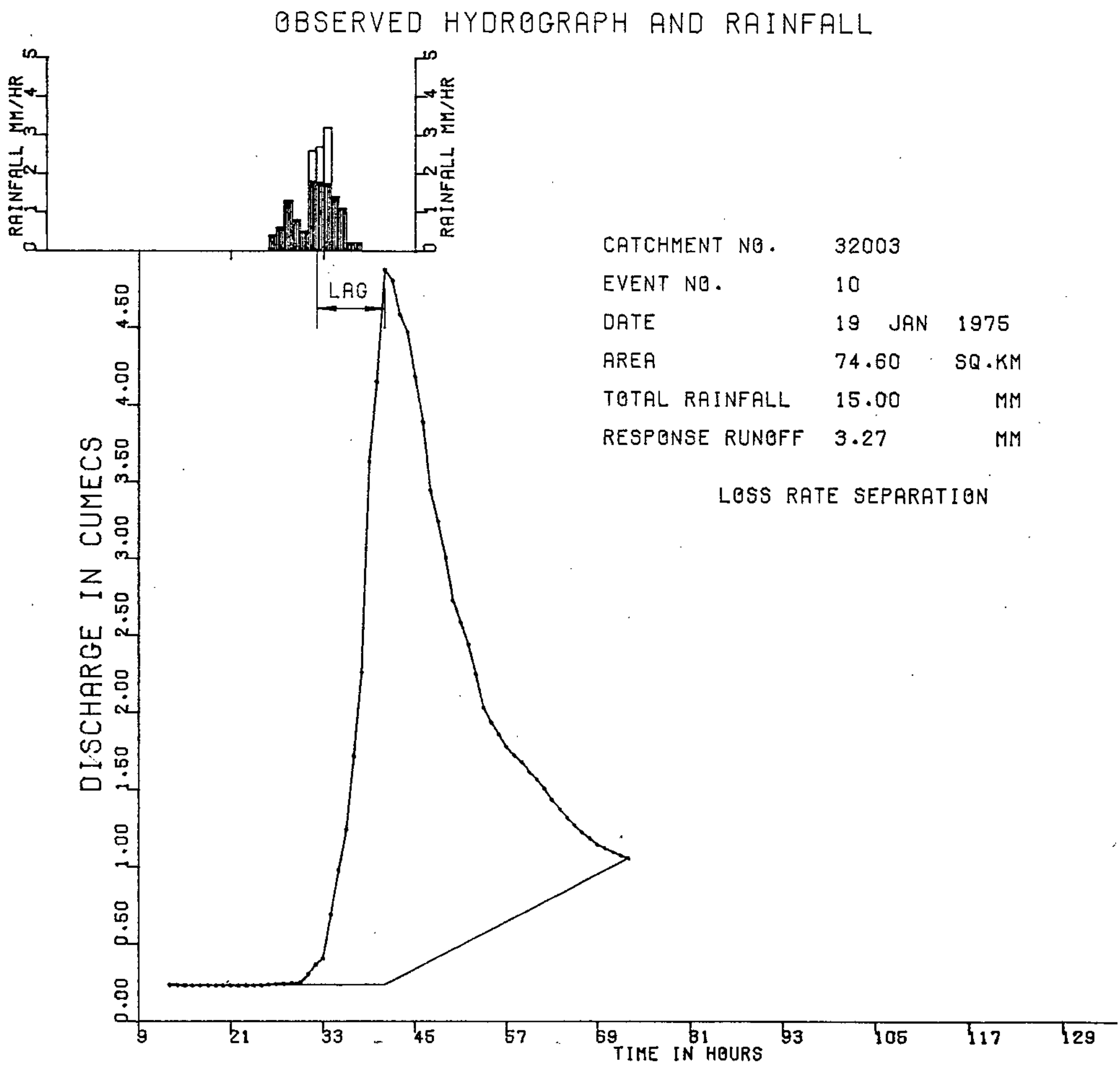
$$CWI = (125 - SMD) + API5 \qquad (1)$$

(iv) The effective rainfall profile is calculated from the total rainfall hyetograph by distributing losses on an hourly basis according to the state of the catchment such that the effective rainfall total equals the volume of runoff. This method takes into account the wetting up of the catchments throughout the storm by permitting smaller losses to be deducted from rain falling later in the storm

(v) The ratio net rainfall/total rainfall or percentage runoff is calculated.

All events then underwent several checking procedures to ensure that the variables listed above were representative of the catchment. Firstly the total and effective rainfall hyetographs and the discharge hydrograph were plotted together for each event, using a standard computer plotting routine. A typical hydrograph is shown in Figure 5. This enabled visual

Figure 5. Typical event plot.



screening of events, to check that the rainfall and flows were compatible and that the baseflow separation was acceptable. Flow hydrographs were examined for any inconsistencies in response, such as a flat topped hydrograph and lower than average runoff, which may be due to the discharge exceeding channel capacity. Daily rainfall percentages were also checked to ensure that the rainfall was spatially uniform and that the catchment average storm rainfall was not biassed by a few extreme values.

Events which passed these checks, and did not exceed channel capacity were included in the study of 'losses' or the volumetric response of the catchment. Details of individual events are presented in Appendix A.

4.2 Unit hydrograph derivation

The separated rapid response hydrograph and net rainfall hyetograph were used to derive a unit hydrograph for each event, using the matrix inversion method with smoothing (FSR Vol 1, 6.4.6). Smoothing is performed using a simple 3 point moving average passed twice through the data.

Events rejected from the losses study, were also rejected from the unit hydrograph study, since errors in percentage runoff would be carried through into the rainfall and flow separations and hence unit hydrograph determination. Of the remaining events, approximately 30% failed to yield a stable and realistic unit hydrograph. The scarcity of autographic rainfall data, especially for the earlier events was felt to be one of the prime causes of unit hydrograph failure. Often the nearest working autographic gauge was outside the catchment boundary and possibly unrepresentative of the rainfall profile over the catchment. In the Nene at Upton (32006) catchment for example, the nearest available gauge for events prior to 1976 was Stanford Reservoir, located outside the catchment boundary, 17.8 kms from the centre. For storms which are relatively stationary, with a uniform rainfall distribution, this gauge will adequately represent catchment conditions. However the presence of localised cells of more intense rainfall or storm movement will lead to an unrepresentative profile and/or timing discrepancies. Two events on this catchment were rejected for this reason.

In view of the inadequacies of the autographic data, the stability of the unit hydrograph was used as a guide to whether the rainfall profile was representative of rainfall over the catchment. Where the unit hydrograph was stable, but did not rise at the origin, the rainfall profile was either advanced or delayed by up to two hours. This provided some allowance for storm movement, and also the uncertainties surrounding the timing of certain raingauges (whether British Standard or Greenwich Mean Time).

Unit hydrograph parameters, namely time to peak in hours (Tp) and peak in cumecs/100 km^2 (Qp) were derived by representing the smoothed unit hydrograph as a triangle, and are summarized in Table 3. This technique was employed in the Flood Studies analyses and enables direct comparison with unit hydrograph parameters estimated from catchment characteristics (see Section 5).

4.3 Catchment average unit hydrographs

The objective of this part of the study was to determine an average unit hydrograph to characterize the response of each of the gauged catchments. Only those events with acceptable unit hydrographs were utilised which ensured that the average would not be biassed by data of doubtful authenticity.

Experience with the national data set, and indeed supported by the differing response displayed by adjacent catchments in the Nene basin, shows that extreme caution is necessary when interpreting apparent trends in unit hydrograph dimensions. For example at Harpers Brook (32003), the two largest of four events tend to have the peakiest unit hydrographs. However four events are clearly insufficient to draw any conclusions from this apparent trend. Therefore unless the evidence is very consistent and compelling, our practice is to adopt a mean value over several hydrographs. It should be noted that in the design case, the effect on the output hydrograph of quite large variations in unit hydrograph shape are much suppressed by the convolution process.

Two methods of deriving an average unit hydrograph were available.

(a) taking a 'by eye' average of the individual event unit hydrographs for a catchment

(b) summation of rainfall and flow data from all events for a catchment, to determine the unit hydrograph directly. (Superposition).

It was felt that as method (a) involved taking an average of individually smoothed unit hydrographs it would result in an unrealistically flat average unit hydrograph. This can be corrected for by aligning peak flows. Method

Table 3 Details of events accepted for losses and unit hydrograph analyses

Catchment	Event No	Date	Peak flow (cumecs)	Percentage runoff (%)	Lag (hours)	UH peak cumecs/ $100km^2$	UH Tp (hours)	Spilling
Upton (2006)	9	18.4.75	12.1	21.6	14.4	18.5	10.9	N/A
	10	30.12.76	16.7	25.8	15.6	13.0	11.2	
	13	23.1.78	14.5	29.5	11.2	13.5	13.0	
	14	26.1.78	16.2	36.4	20.3	10.0	11.0	
	16	26.5.79	15.2	29.5	14.3	14.5	10.0	
	17	28.6.73	12.0	9.0	15.5	16.0	11.0	
	18	17.11.74	16.0	23.6	12.2	-	-	
	19	19.1.75	13.0	22.1	8.0	17.7	7.5	
	21	23.12.75	7.6	29.4	22.9	-	-	
Andrews (2007)	3	27.6.73	16.4	23.0	23.7	-	-	No
	5	17.11.74	14.0	32.3	17.7	14.4	14.0	No
	6	19.1.75	11.3	19.7	13.6	16.0	11.6	Yes
	8	18.4.75	14.2	23.4	13.0	15.8	12.3	Yes
	9	31.12.76	13.8	32.1	15.7	12.7	17.0	No
	11	23.12.78	7.7	34.5	23.6	-	-	No
...e Brook (2004)	3	17.11.74	13.3	32.5	22.7	-	-	No
	5	19.1.75	8.3	30.2	15.6	8.1 *	17.0*	Yes
	7	18.4.75	15.6	32.1	14.3	9.6 *	15.0*	Yes
	8	30.12.76	12.7	37.5	13.8	7.2 *	18.0*	Yes
	14	7.4.79	14.1	38.8	22.2	6.1 *	20.5*	Yes
Harpers Brook (32003)	6	27.6.73	10.9	35.7	16.3	-	-	N/A
	10	19.1.75	4.9	21.8	8.9	18.5	10.0	
	11	8.3.75	20.5	58.6	8.2	-	-	
	12	18.4.75	19.6	54.3	9.3	27.5	10.5	
	13	31.12.76	15.9	58.9	8.2	-	-	
	14	27.1.77	6.9	34.3	8.8	20.0	9.0	
	18	4.5.78	17.2	50.9	8.2	28.4	5.1	
	19	23.12.78	4.9	26.6	16.0	-	-	
	20	7.4.79	11.4	46.0	14.5	-	-	
Willow Brook (32002)	3	27.6.73	6.5	12.8	25.3	12.1	24.7	N/A
	6	17.11.74	4.4	22.7	17.2	10.6	19.0	
	11	30.12.76	6.7	30.7	23.2	9.3	15.5	
	12	27.1.77	6.7	25.4	16.4	11.5	15.2	
	18	26.5.79	3.0	13.6	16.0	10.6	15.8	

* Qp and Tp derived directly from smoothed UH, as cannot be adequately represented by a triangle

(b) enables the average unit hydrograph to be obtained more directly, and was adopted for this study. All events on a catchment are superposed and a unit hydrograph derived directly from this enlarged event. This involves first aligning the peak net rainfall ordinates for each event, then summing all rainfall and flow ordinates respectively in each time increment. The resultant unit hydrograph tends to be weighted towards the larger events, which has advantages for design applications.

Figures 6 to 15 show the individual event unit hydrographs and catchment average unit hydrograph respectively for each of the five gauged catchments. The results of the unit hydrograph and losses studies are summarized in Table 3, but a few comments on each catchment may help clarify the decisions taken.

4.4 Individual catchment response

(i) Nene at Upton (32006) - Figures 6,7

The flood hydrographs at Upton characteristically show a rapid rise then a flattening off towards the peak commencing at discharges of 17 cumecs. There is some doubt as to whether this flattening results from bypassing or from flood plain storage upstream. A recent flood (14.8.80) of 16.4 cumecs was recorded without any bypassing of the gauging station. However, five out of eleven of the annual peak discharges since 1968, are in the region of 19 cumecs, without exceeding it (see Table 11). This suggests that water may be lost to flood plain storage at discharges exceeding 17 cumecs.

In view of this uncertainty two events with peak discharges of 18.5 cumecs were rejected from the unit hydrograph and losses study. It is recommended that if the peak of any predicted hydrograph (in design or calibration) exceeds 17 cumecs, then the peak should be truncated at this discharge and the excess volume distributed on the recession in accordance with observed events. Examination of the seven accepted unit hydrographs (see Figure 6) shows that there is a tendency for unit hydrograph peaks to decrease with the size of the event (as indexed by the observed peak discharge). This suggests that water may be entering flood plain storage upstream of the gauging station. It will also be reflected in the average unit hydrograph for the catchment.

Figure 6. FSR unit hydrographs-R.Nene at Upton (32006)

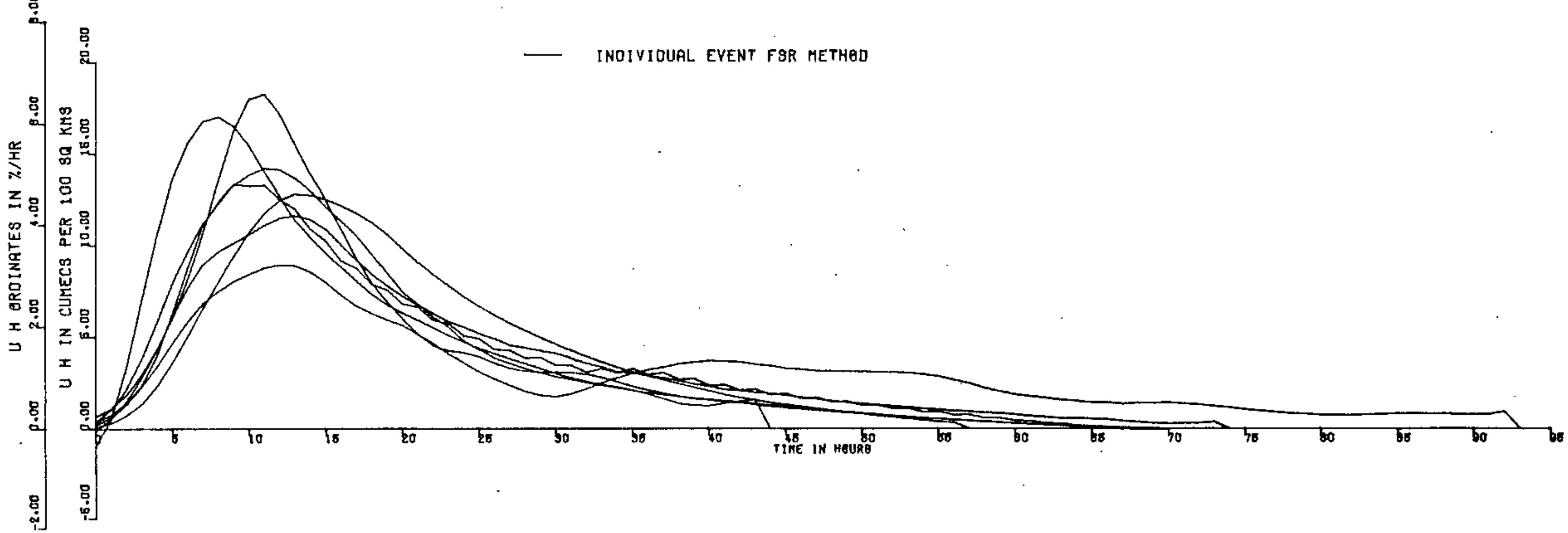
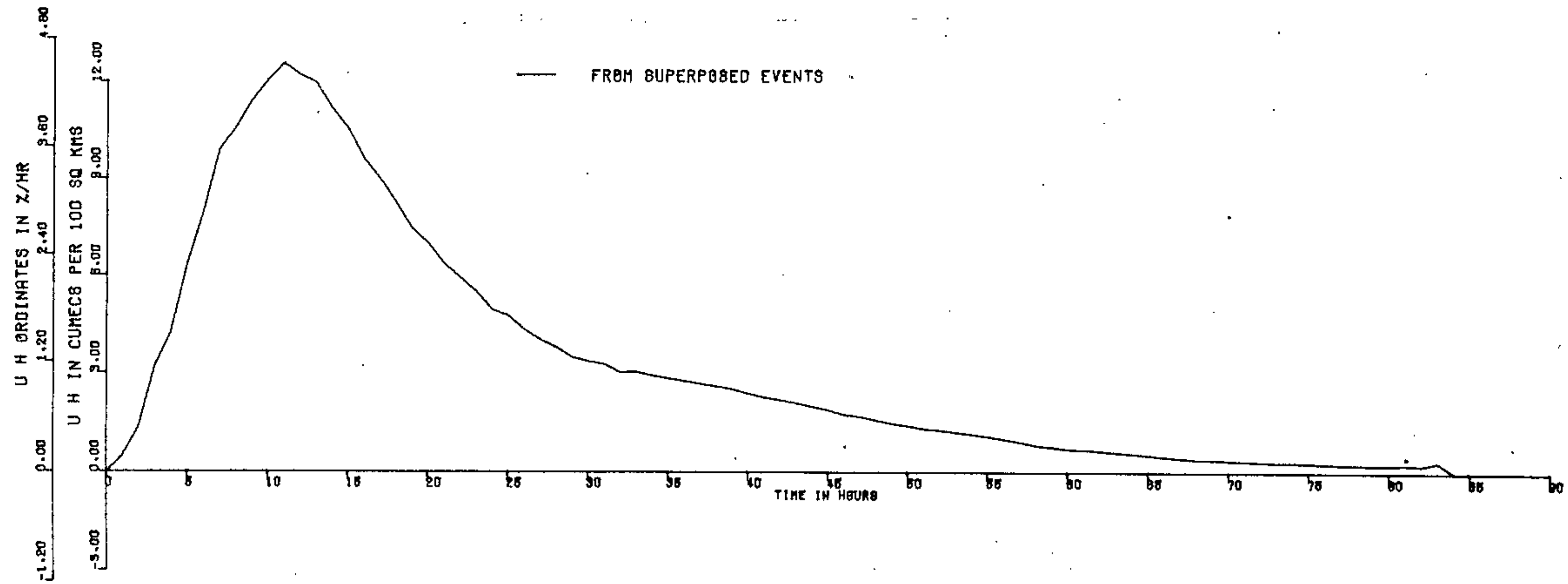


Figure 7. Average unit hydrograph - R.Nene at Upton



(ii) Nene at St Andrews (32007) - Figures 8,9

Approximately 30% of this catchment contributes runoff to three reservoirs at Hollowell, Pitsford and Ravensthorpe. All these reservoirs have adequate capacity and during most flood events are filling, rather than spilling. Each event in Table 3 is indexed as to the reservoir state. The reservoirs were full and spilling for only two of the four events. When reservoirs are filling the catchment upstream of the reservoir does not contribute to flow at the gauging station, so percentage runoffs were calculated on the basis of this reduced contributing area. The two events with the reservoirs spilling have slightly peakier unit hydrographs and shorter response times than the 'no spill' hydrographs. However events with reservoirs spilling tend to occur after a prolonged wet period, and the peakier unit hydrographs may reflect this. Derivation of separate 'spill' and 'no spill' unit hydrographs is therefore unwarranted.

(iii) Ise Brook at Harrowden (32004) - Figures 10,11

Flow hydrographs for this catchment are typically irregular with a tendency for a slight double peak of overall duration twenty hours or so. This reflects the initial rapid runoff contribution of Slade Brook (a major tributary draining the urban area around Kettering), followed by the slower attenuated response of the upper reaches. Baseflow levels are also very variable, as a result of abstractions and controls on flow, and smoothing of low flows was often necessary to achieve a realistic baseflow separation. These factors make it a difficult catchment to model using unit hydrograph techniques.

Approximately 6% of the catchment, contributes to Thorpe Malsor and Cransley reservoirs, located on adjacent tributaries of the Ise. Events are indexed with the state of the reservoirs, as shown in Table 3. It can be seen that for most flood events the reservoirs are spilling, so a 'spilling' average unit hydrograph will be most typical of catchment conditions.

(iv) Harpers Brook at Old Mill Bridge (32003) - Figures 12,13

The flashy nature of this catchment should make it ideally suited to unit hydrograph analyses. However only four of the nine events examined yielded acceptable unit hydrographs, due largely to the lack of an

Figure 8. FSR unit hydrographs – River Nene at St Andrews (32007)

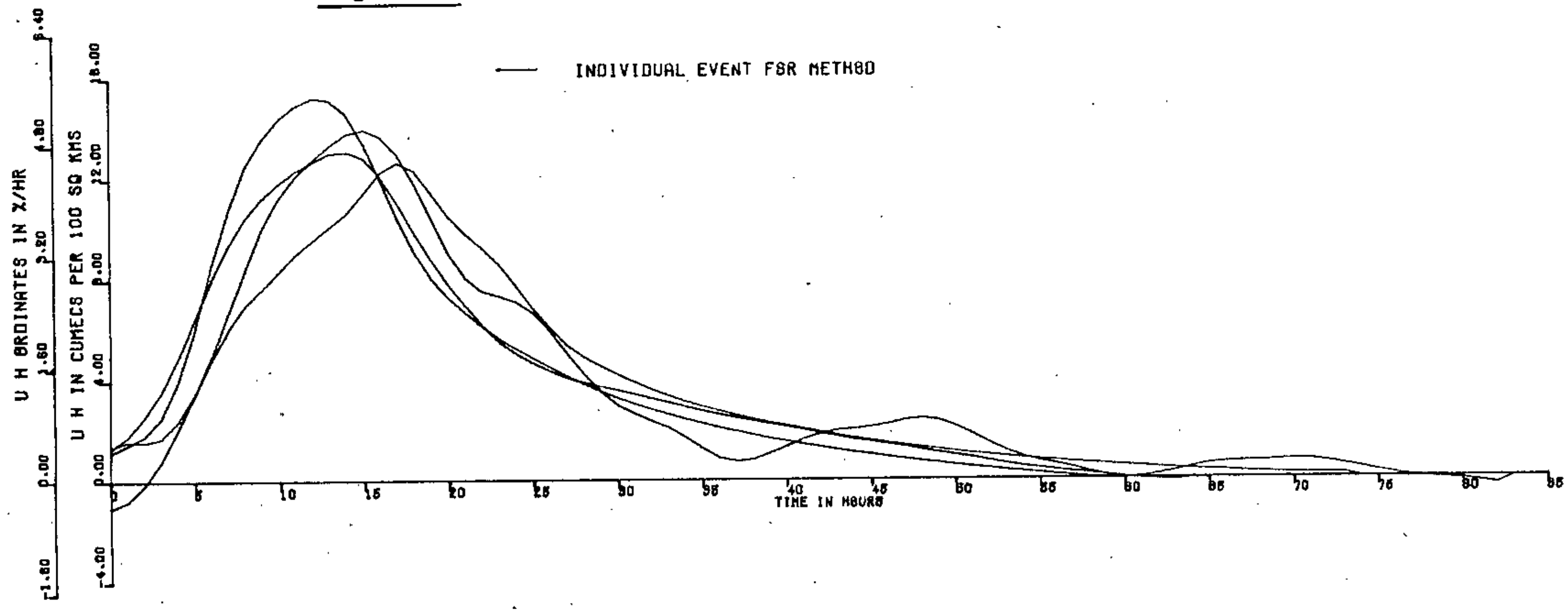


Figure 9. Average unit hydrograph – River Nene at St Andrews

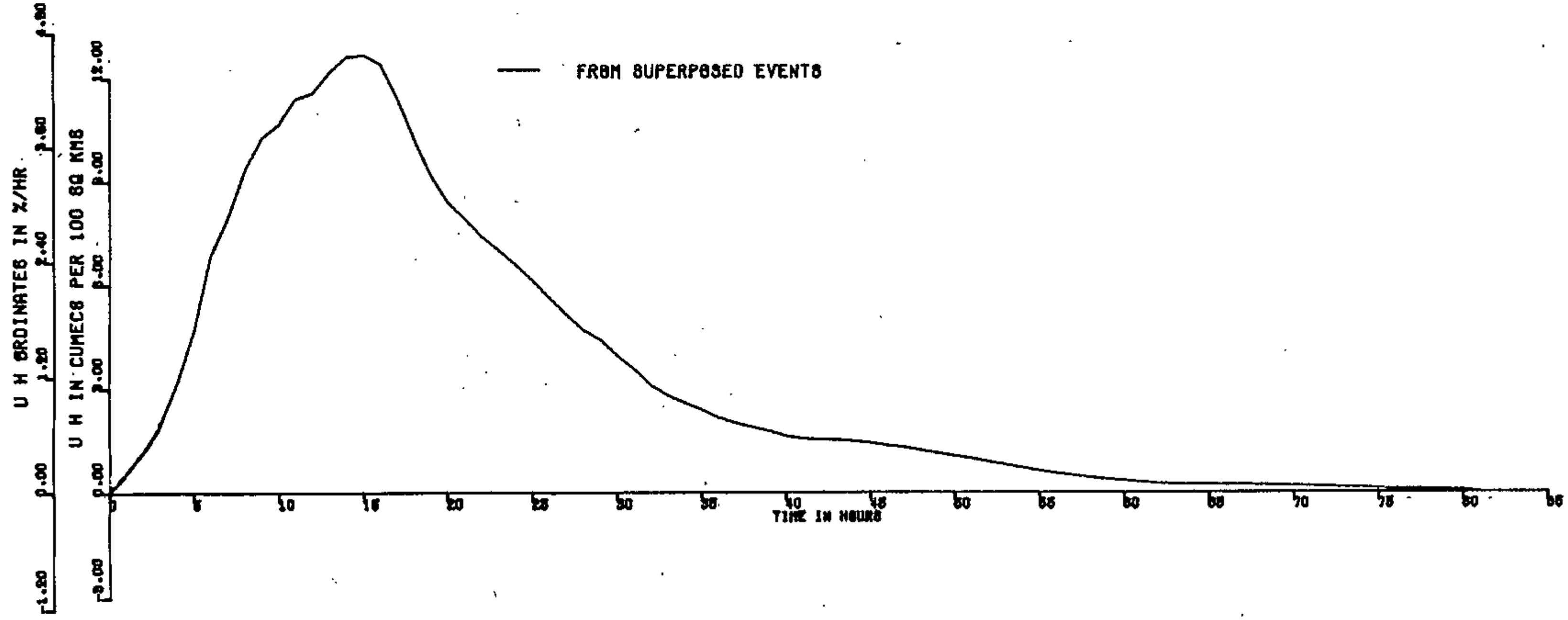


Figure 10. FSR unit hydrographs - Ise Brook at Harrowden (32004)

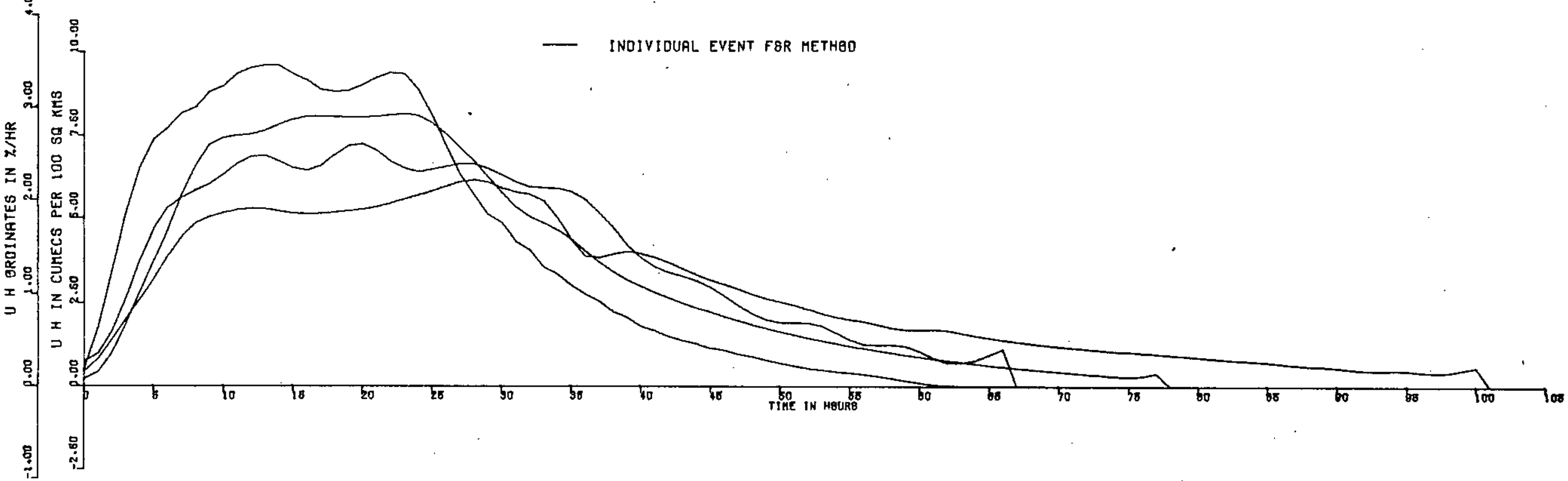


Figure 11. Average unit hydrograph - Ise Brook at Harrowden

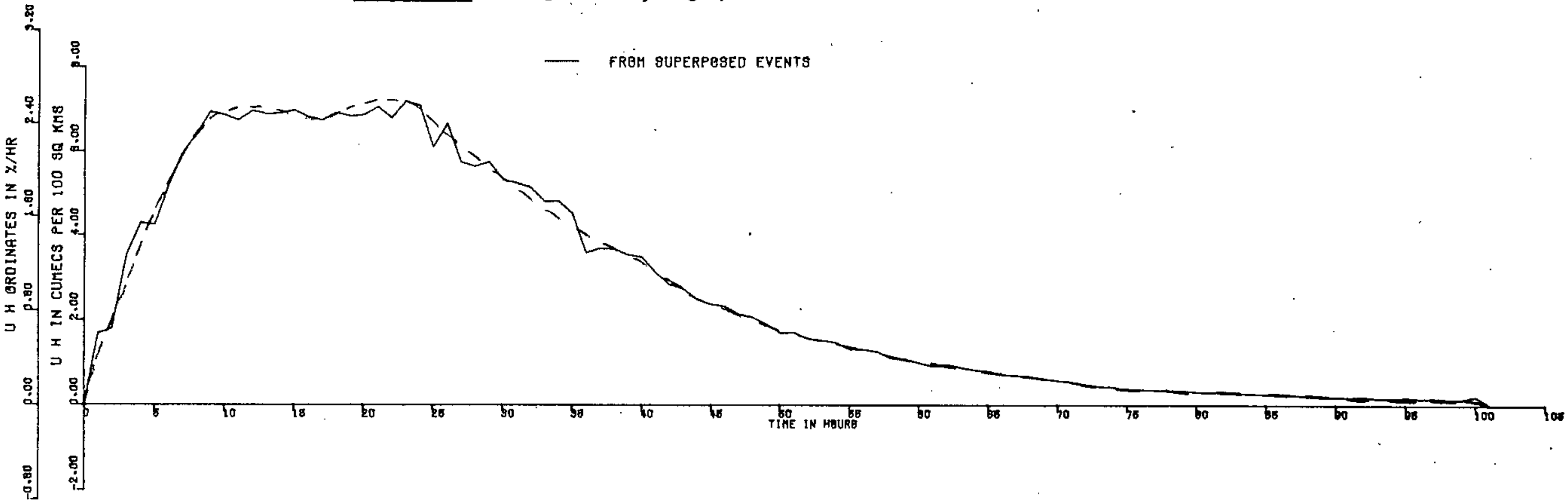


Figure 12. FSR unit hydrographs - Harpers Brook at Old Mill (32003)

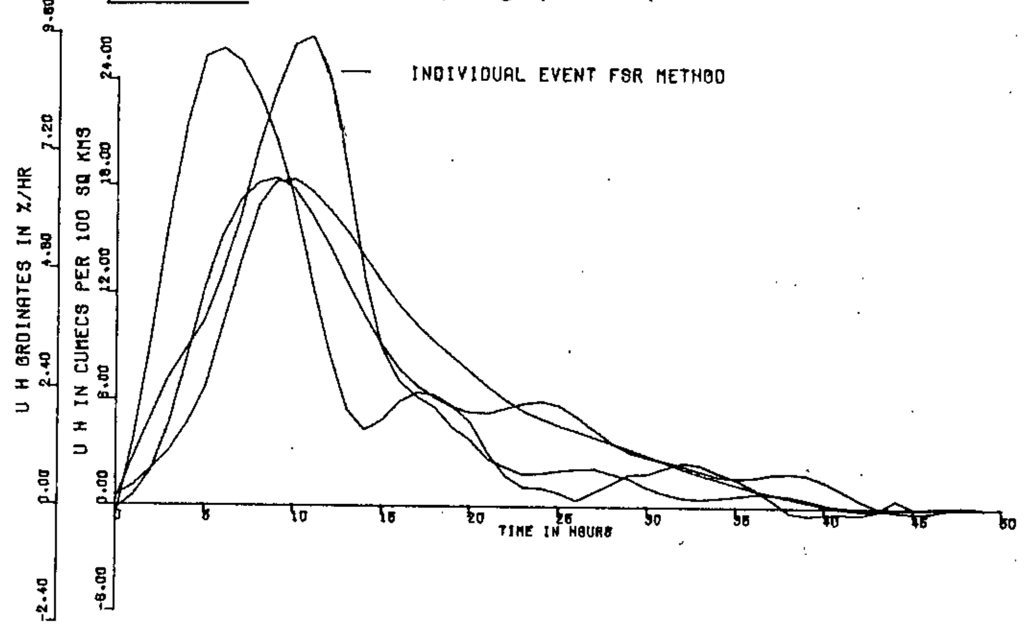
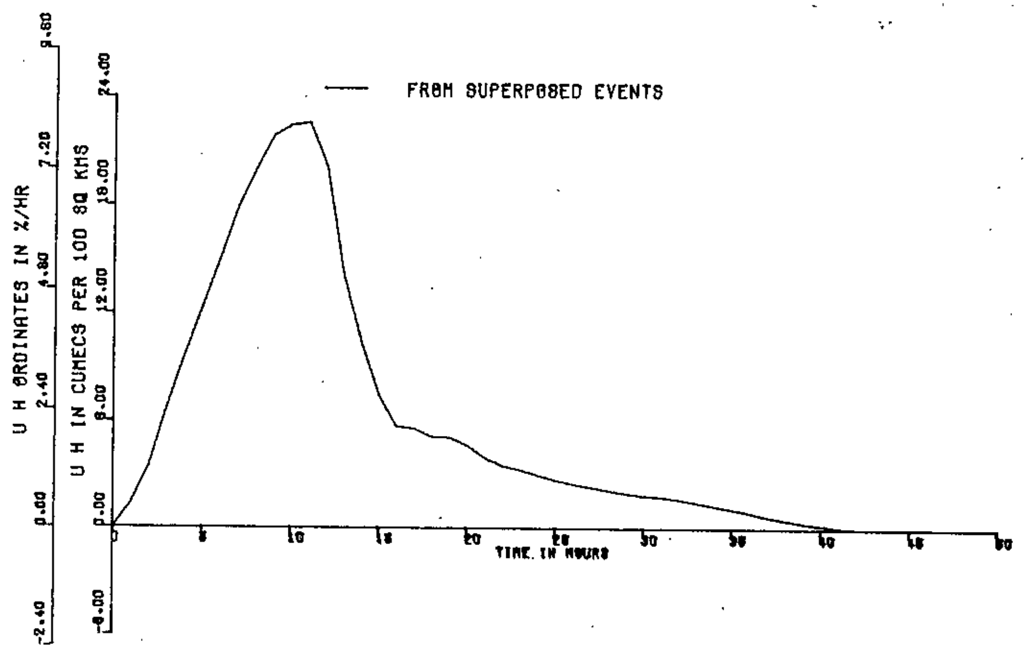


Figure 13. Average unit hydrograph - Harpers Brook at Old Mill



autographic gauge centrally placed on the catchment. Percentage runoff data showed a slightly non linear response with larger events tending to produce higher percentage runoffs. The data set is however biassed towards large events, so an average will be applicable to the design case.

(v) Willow Brook at Fotheringhay (32002) - Figures 14,15

This is rather a difficult catchment to model using unit hydrograph techniques, due to the typically rather irregular and sluggish response. This may be attributed to the effects of artificial controls on flow, as the river drains through several ornamental lakes and ponds. Abstractions also cause sudden drops in level, which were smoothed out prior to the analysis, with care taken to maintain the natural recession.

Figure 14. FSR unit hydrographs - Willow Brook at Fotheringhay (32002)

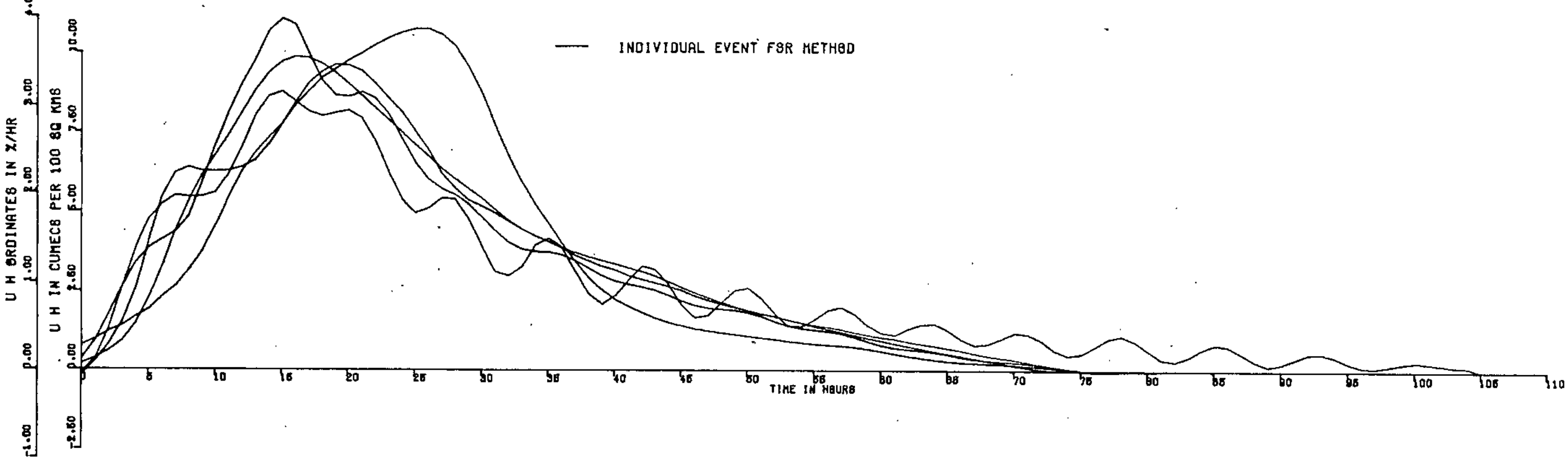
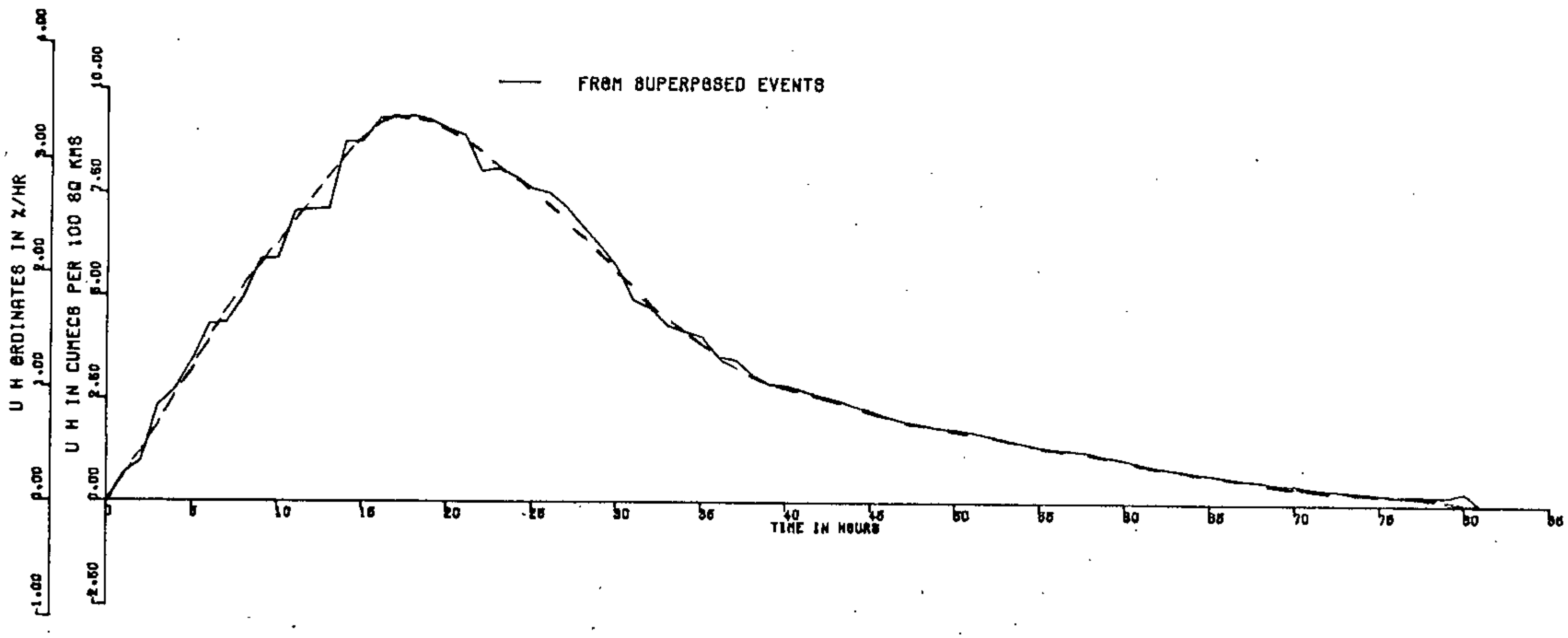


Figure 15. Average unit hydrograph - Willow Brook at Fotheringhay



5.0 UNGAUGED CATCHMENTS

47% of the catchment upstream of Wansford is ungauged for flood flows. The most realistic way of modelling this large and fairly diverse area was as a series of distributed inputs along the length of the Nene. The ungauged areas were therefore divided up into 23 subcatchments, as shown in Figure 16, ranging in size from 16 to 111 km^2 and each based on a tributary of the Nene. A certain amount of grouping of catchments and inclusion of areas immediately adjacent to the river into the nearest subcatchment, was necessary in order to reduce the computational work to manageable proportions. Also it was felt that 23 subcatchments would provide the level of accuracy required by the Nene model.

Where the timing or response of small, adjacent tributaries was similar their catchment areas were combined for the purposes of hydrograph prediction such that Tp is based on a single catchment, and the volume of runoff based on the combined areas.

The next two sections describe how standard percentage runoff and timing of response were estimated at ungauged sites in the Nene.

5.1 Standard Percentage Runoff

At the ungauged site the standard percentage runoff can be estimated using the soil index and fraction of urban development

$$SPR = 95.5\,SOIL + 12\,URB \qquad (2)$$

Since SPR is entirely dependent on the permeability of the ground surface, it is constant for a catchment. Dynamic considerations such as the amount of rainfall and state of the catchment are included at a later stage when SPR is converted to the actual percentage runoff for an event.

The SOIL index is derived from the five soil classes shown on the FSR winter rainfall acceptance potential map. This index was calculated for each Nene subcatchment as follows,

$$SOIL = 0.15S_1 + 0.30S_2 + 0.40S_3 + 0.45S_4 + 0.50S_5 \qquad (3)$$

where S_1 - S_5 are the percentages of the catchment area on a given soil type. Table 4 shows the mean observed SPR calculated from the hydrograph analysis (Section 4.1) and the SPR estimated from SOIL. Three catchments (32002, 32003, 32004) show good agreement between observed and predicted, but 32007 and 32006

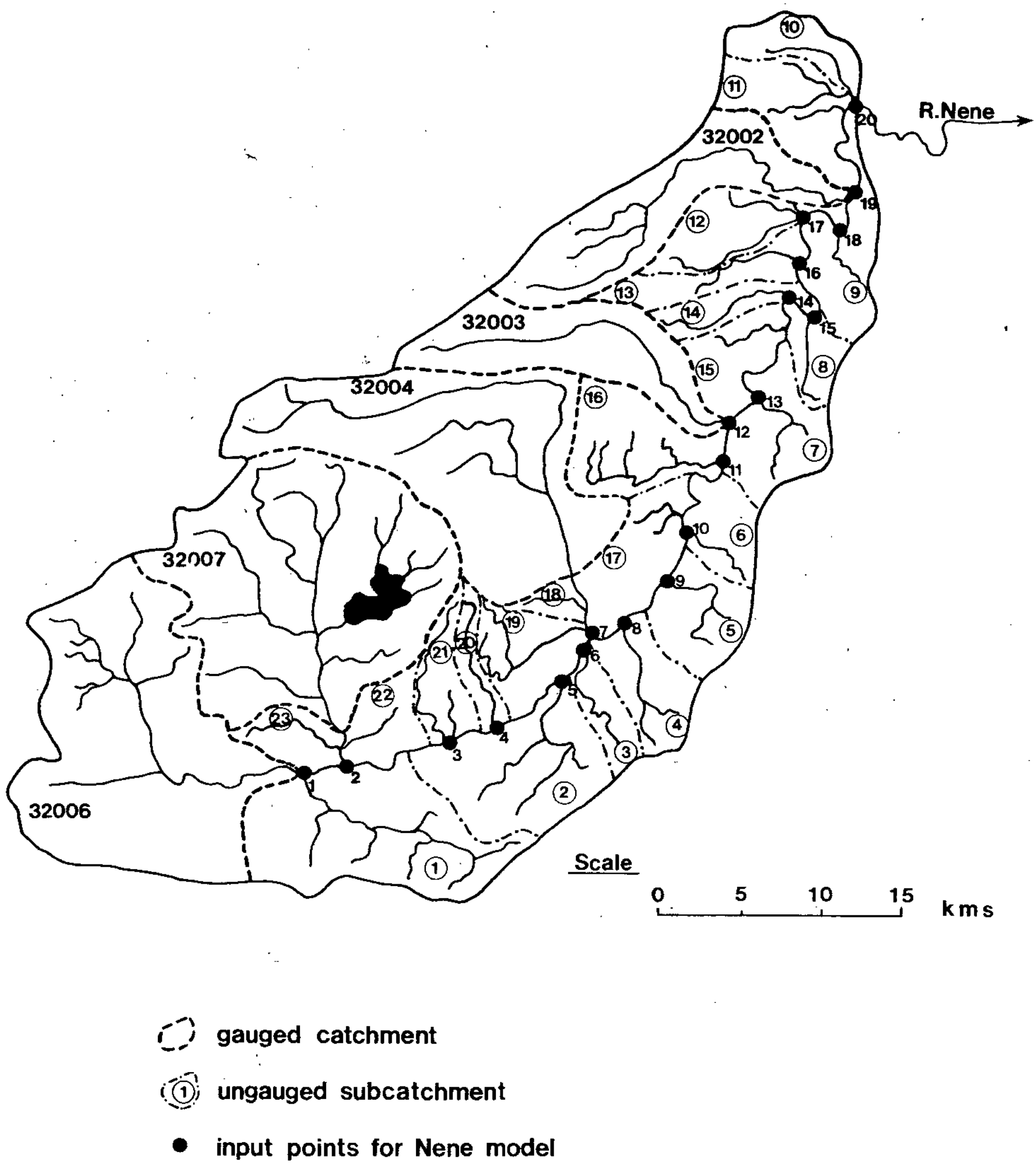


Figure 16. Location of ungauged catchments and hydrograph input points for River Nene model.

are over predicted by 20% and 17% (in SPR units) respectively. These errors are primarily due to the difficulties of mapping soils on a regional basis and of grouping their hydrological response into five classes. One approach of reducing this error is to use local SPR values and modify the SOIL based estimates accordingly. This approach is difficult to justify in the Nene area where there was no rational basis for distinguishing the well predicted from the poorly predicted soil types.

An alternative approach developed for this study has been to develop a national relationship between SPR and the base flow index, BFI*. Full use of the large number of gauged catchments (only some of which are suitable for unit hydrograph analysis) in the Nene is then made by relating the gauged BFI values to catchment solid and drift geology. The inclusion of this additional data which provides information on the nature of the response, enables SPR to be predicted from gauged BFI or from catchment geology at sites where unit hydrograph analysis was not possible. The method is summarised below.

The Base Flow Index (BFI) is calculated from the mean daily flow hydrograph using programmed separation rules. The index was devised primarily to relate to low flows, but given that (1 - BFI) is a measure of the quick response proportion of the hydrograph it may be expected that BFI will relate to the flood characteristics of the river such as Standard Percentage Runoff (SPR). In comparing (1 - BFI) with SPR it must be remembered that the former is an index of the proportion of the separated to total discharge; the latter the proportion of separated runoff to rainfall. The separation procedures are also different and BFI is based on the annual hydrograph of daily flows, SPR is based on hourly event data.

Figure 17 shows the relationship between BFI and SPR estimated from flow data for 104 catchments in the UK. and demonstrates that the five Nene catchments are in close agreement with the National relationship.
The equation of the line is

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

and thus predicts an SPR = 0 when BFI = .98. This is supported by individual catchments hydrographs where the maximum observed BFI is 0.98, which only occurs on drift free permeable chalk catchments which have very little quick response runoff. Equation (4) may be compared with the following equation based on the same 104 catchments which relates SPR to SOIL from the winter rainfall acceptance map.

* Low Flow Studies Report, NERC (1980)

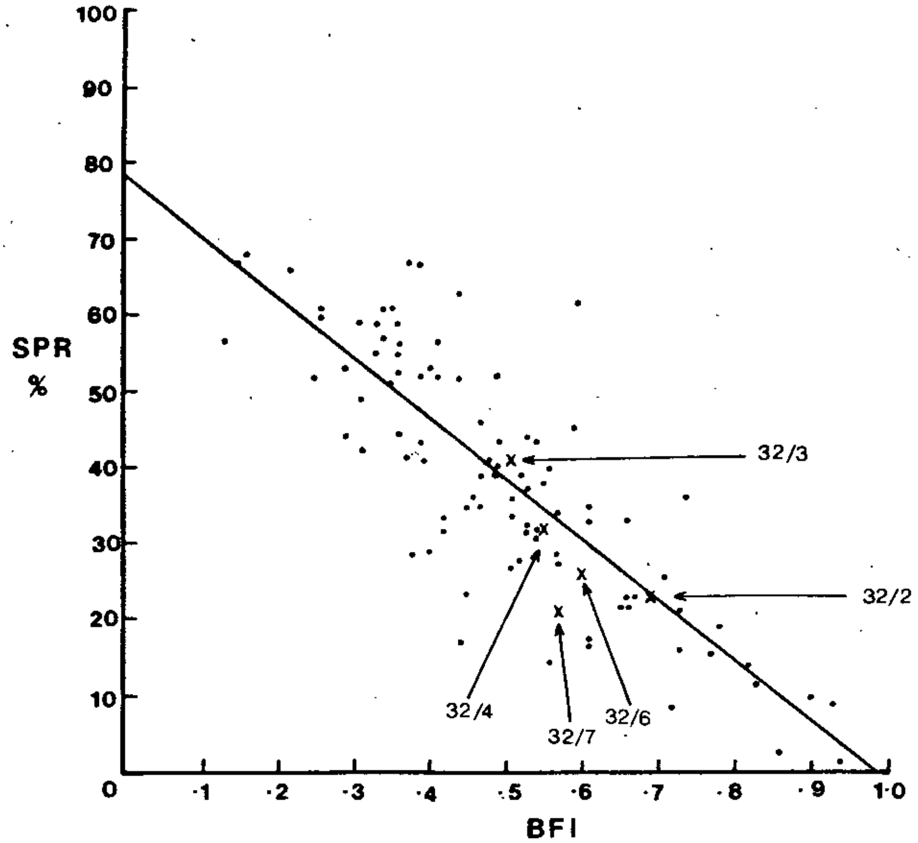


Figure 17. National SPR-BFI relationship

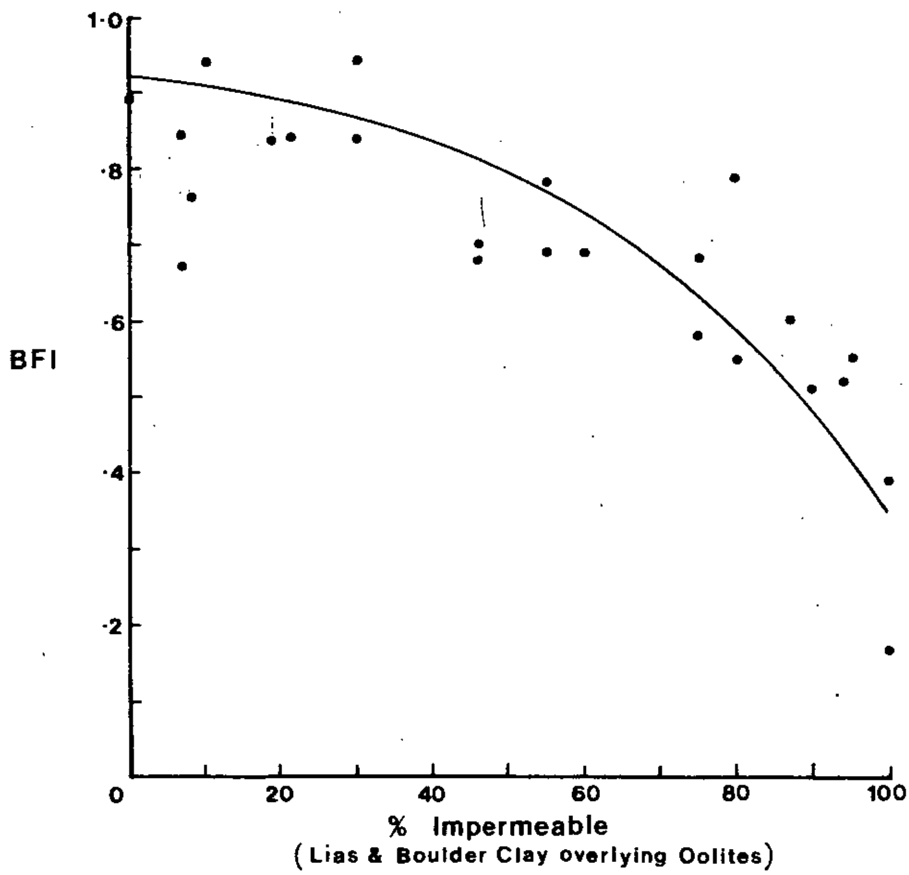


Figure 18. Relationship between BFI and % impermeable for Oolite catchments

$$SPR = 122.1\ SOIL - 7.6 \quad se = 11.54 \quad r^2 = 0.50 \qquad (5)$$

This would suggest that if SPR is to be estimated on a catchment which has a flow record it would be better to use BFI calculated from this record rather that the soil map (assuming unit hydrograph analysis cannot be carried out). Figure 17 also shows the SPR, BFI data for local catchments on the Lias, Oxford Clay and on the Oolites - they tend to support the national data set and would not justify using a local SPR-BFI equation.

On ungauged catchments BFI is estimated from catchment geology as follows:-

(i) Figure 18 shows the relationship between BFI and the percentage of impermeable geology for catchments with a solid geology of Inferior or Great Oolitic Limestone. The percentage of impermeable cover is the areal proportion of Boulder Clay overlying Limestone, plus the proportion of Lias multiplied by 100. The line shown in Figure 18 has been drawn by eye, the error of estimating BFI is estimated as approximately 0.05.

(ii) On ungauged Lias catchments a BFI of 0.47 should be used provided the proportion of Middle Lias is less than 25%. There was no relationship between the proportion of Boulder Clay and BFI for Lias catchments (Figure 19). There is a weak positive correlation (0.49) between BFI and the proportion of Middle Lias for Lias catchments. The Middle Lias is a more permeable formation than the Upper or Lower Lias and Figure 20 should be used for estimating BFI if a Lias catchment contains more than about 25% Middle Lias.

(iii) On ungauged Boulder clay catchments a BFI of 0.41 should be used. This is estimated from the mean of local Boulder Clay catchments which were free of sand and gravel. The value of .41 should only be used for catchments dominated by Boulder Clay (> 90%). In other cases Figure 18, Figure 20 or the value of .47 for Lias catchments should be used, as appropriate.

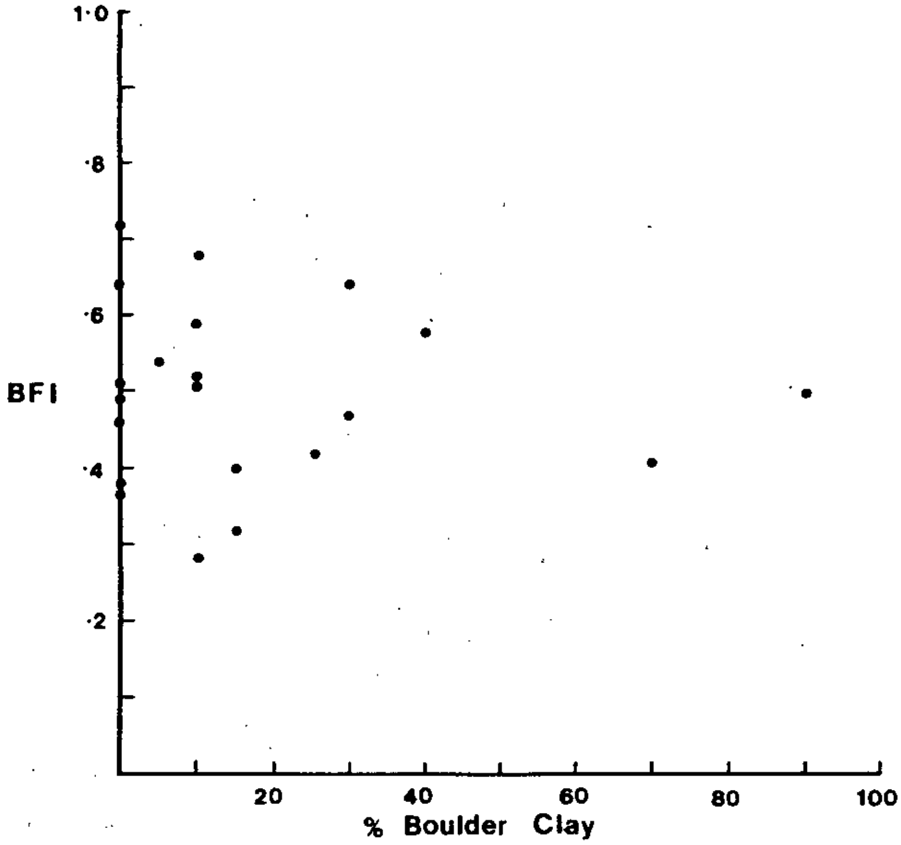


Figure 19. Relationship between BFI and % Boulder Clay for Lias catchments

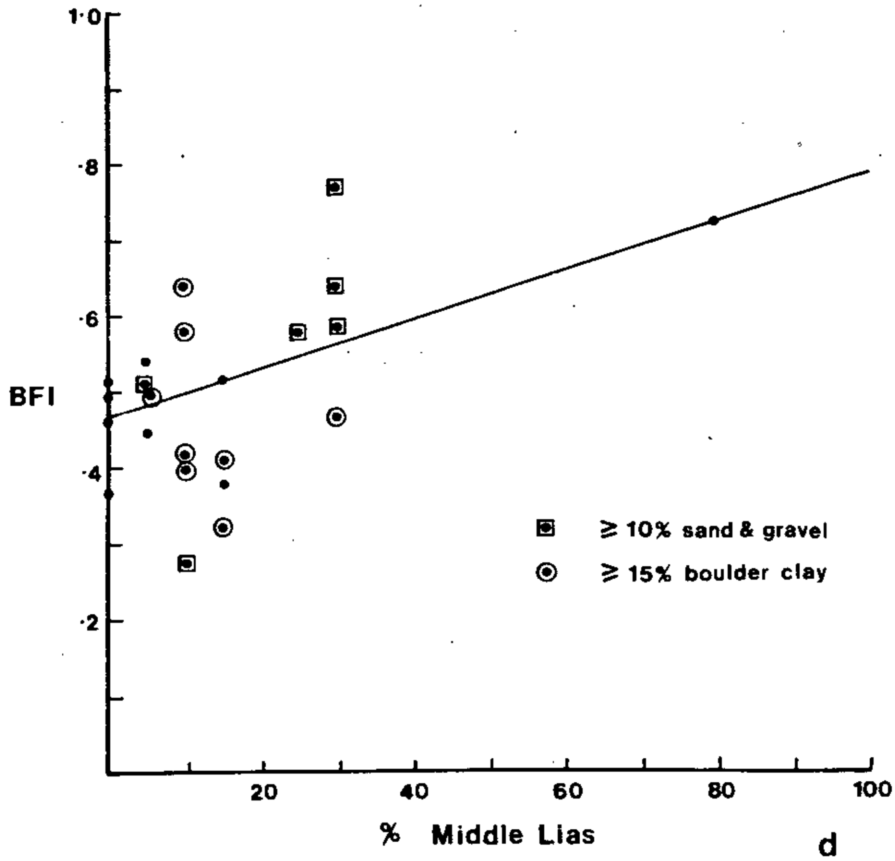


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Figure 20. Relationship between BFI and % Middle Lias for Lias catchments

(iv) On ungauged Oxford Clay catchments the BFI should be estimated using Figure 21 which plots the BFI against the proportion of Oxford Clay.

For catchments on 100% of a given solid or drift geology the following BFIs should therefore be used. The corresponding SPR is calculated from equation 4.

	BFI	SPR %
100 % Oolites	.92	5
100 % Lias	.47	41
100 % Boulder Clay	.41	46
100 % Oxford Clay	.26	57

It should be noted that this enables SPR values to be predicted outside the range of those estimated from the soil index. It can be seen from Figure 17 that a number of catchments do display SPR values beyond those predicted for non urbanized catchments. This is of importance in this area where both Oolitic Limestone and Oxford Clay represent the extremes of the response scale.

Table 4 compares the mean observed SPR (from hydrograph analyses) with (i) SPR estimated from SOIL and (ii) SPR estimated from BFI using equation 4. The observed BFI, calculated from mean daily flow data, and the BFI estimated from geology are also compared for the five gauged catchments. For three catchments the estimates from SOIL and both BFI estimates are within 6% (in SPR units) of the observed. For the two poorly predicted catchments, both BFI estimates are better than the SOIL based ones.

Table 5 shows the SPR values for each ungauged subcatchment estimated from SOIL and BFI. The estimates of BFI are based on catchment geology, but where appropriate the observed BFI's from adjacent gauging stations are incorporated. These estimates are marked with an asterisk to indicate that greater confidence should be placed on them than on SOIL based estimates. In general for large catchment areas the difference between the two approaches is small. However for small catchments where local differences in solid and drift geology can be used directly to estimate SPR, the BFI estimates are to be preferred.

For the two urbanized subcatchments (numbered 22 and 23) in Northampton, SPR was estimated by a different procedure, which considers the contributions from rural and urban areas separately (FSR Supplementary Report No. 5).

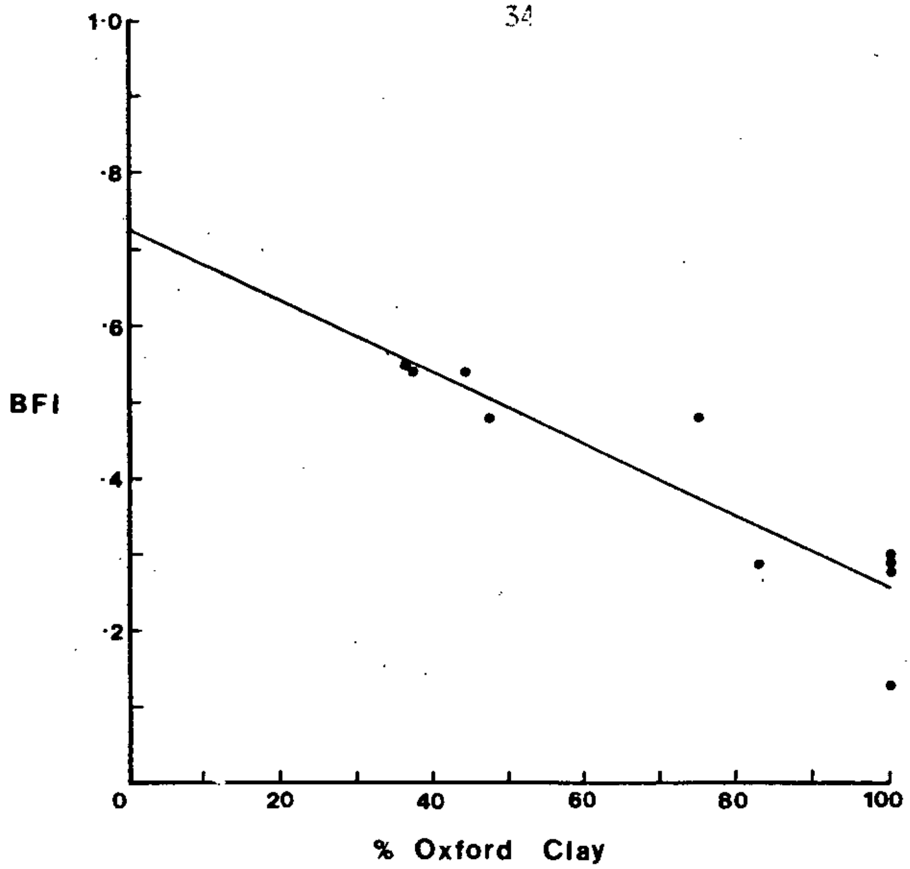


Figure 21. **Relationship between BFI and proportion of Oxford Clay in a catchment**

Catchment	STANDARD PERCENTAGE RUNOFF derived from				BFI	
	Observed Floods	Soil Map	BFI from flow data	BFI estimated from geology	Observed	Estimated from Geology
32002	23	25	23	17	.69	.77
32003	41	43	38	40	.51	.48
32004	32	31	34	30	.55	.61
32006	26	43	30	33	.60	.57
32007	26	41	32	28	.57	.63

Table 4 Comparisons of SPR derived from observed data, SOIL, BFI and geology and BFI comparison

Table 5 Catchment Characteristics for Ungauged Catchments

No	Area (km^2)	Length (km)	S1085 (m/km)	Urban	% Lake	Soil	SAAR (mms)	RSMD (mms)	SPR % (Soil)	SPR % (geology)	BFI (geology)
1	110.92	17.9	2.02	.03	0.0	0.434	630	22.0	41.8	37 *	.52
2	66.44	13.5	4.21	.02	0.25	0.432	610	21.9	41.5	30 *	.60
3	15.80	10.5	6.46	.06	0.0	0.436	595	23.2	42.4	31 *	.59
4	29.75	9.3	6.21	.035	0.5	0.437	595	23.1	42.1	29	.62
5	31.0	8.6	6.29	.187	1.3	0.436	602	22.9	43.5	37	.52
6	26.9	6.7	4.97	.08	0.0	0.422	601	22.7	41.3	39	.49
7	39.4	7.7	6.60	.032	2.0	0.380	598	22.7	36.7	47	.39
8	20.75	5.3	6.60	.006	0.0	0.368	600	23.8	35.2	48	.38
9	40.37	4.0	5.18	.025	0.1	0.263	585	21.5	25.4	45	.42
10	24.20	9.4	8.21	.021	0.4	0.150	580	23.1	14.6	12 *	.83
11	40.20	5.4	9.41	.041	0.3	0.150	635	22.3	14.8	9 *	.87
12	31.41	9.8	7.86	.004	0.0	0.328	618	22.9	31.4	25 *	.67
13	22.50	12.35	6.58	.019	0.0	0.316	625	23.5	30.4	24 *	.68
14	21.08	9.92	5.78	.056	0.13	0.325	617	24.0	31.7	42	.46
15	28.51	7.47	6.53	.016	0.0	0.329	598	23.6	31.6	42	.46
16	43.94	14.95	4.78	.019	0.0	0.297	612	23.4	28.6	15	.79
17	36.04	4.8	12.69	.045	0.0	0.373	598	22.0	36.2	22	.70
18	12.05	5.25	10.99	.036	0.0	0.165	615	24.7	16.2	33	.57
19	30.15	13.50	6.86	.22	0.0	0.414	615	23.9	42.2	35	.54
20	16.0	9.6	9.44	.054	1.95	0.339	618	24.7	33.0	29	.62
21	20.82	11.22	7.35	.024	2.7	0.348	620	24.6	33.5	24	.68
22	18.75	4.40	12.84	.560	2.0	0.33	615	23.9	38.3	-	-
23	16.75	8.10	7.33	.42	0.6	0.39	615	23.9	42.1	-	-

5.2 Synthetic Unit Hydrograph Derivation

Where there was no flood data available it was necessary to derive synthetic unit hydrographs based on catchment characteristics. These could then be convoluted with any rainfall event and baseflow added to yield an inflow hydrograph to the Nene model. Several catchments are gauged at low flows only, and this data was incorporated where possible.

Catchment characteristics, namely length (L), slope(S1085) and proportions of the catchment affected by urban (URBAN) and lake (LAKE) were derived from 1:25,000 scale Ordnance Survey maps (see FSR Vol.I, 4.2). Others, namely the soil (SOIL) and climate factors (RSMD) were calculated using parameters extracted from maps contained in the Flood Studies Report. These values are summarized in Table 5.

The 1 hour unit hydrograph from 10 mm of rainfall was represented on each ungauged subcatchment as a simple triangle. The shape of this triangle is controlled by the time to peak (T_p), which may be calculated from catchment characteristics (Vol 1, 6.5.4) as follows.

$$T_p = 46.6 L^{0.14} \; S1085^{-0.38} \; (1 + URBAN)^{-1.99} \; RSMD^{-0.4} \qquad (6)$$

T_p values for the ungauged catchments are given in Table 9. Once T_p is known, the other unit hydrograph parameters may be calculated. Q_p, the peak of the unit hydrograph in cumecs/100 km^2 is given as

$$Q_p = 220/T_p \qquad (7)$$

and the baselength (forced so that the triangle contains 10 mm of runoff) as

$$TB = 2.52 \; T_p \qquad (8)$$

If a different data interval to 1 hour is utilised, the unit hydrograph can be modified by adjusting T_p to T'_p and hence recalculating the other parameters.

$$T'_p = T_p + (T - 1)/2 \text{ where T is the data interval} \qquad (9)$$

From the 3 parameter values, Q_p, T_p and TB the unit hydrograph shape can be defined and ordinate values at the specified data interval extracted by interpolation.

Incorporation of local data

Since predictions of unit hydrograph shape are heavily reliant on time to peak (T_p) estimates, it was decided to assess, as far as possible, the applicability of the prediction equation for T_p (equation 6) to the Nene area. Observed and predicted T_p values were therefore compared for the five gauged catchments and as many neighbouring catchments as possible. Catchment characteristics for the five gauged tributaries are summarized in Table 6. The observed T_p for a catchment was derived by fitting a triangle to each event unit hydrograph, and taking the mean of the individual T_p's. To extend the data set, events from low flow gauging stations within the catchment were also included. The only available data for these events is the time of peak flow and the centroid of the rainfall profile, as taken from the nearest autographic gauge. The difference between these times is the catchment lag and enables calculation of T_p (T_p = 0.9 LAG).

As indicated in Table 7 the residuals between observed and predicted T_p's range from 4 to 0.6 hours, but are on average between 1 - 2 hours. The prediction equation makes no allowance for lake storage and it's attenuating effect on flood flows. It was therefore not surprising that for the St Andrews catchment (30% reservoired), T_p was under-predicted by 3.0 hours. Ise Brook and Willow Brook catchments also had longer response times than predicted, which again may reflect lake storage.

Overall the range of residual values was to be expected considering the large variability in T_p values between events on the same catchment, and the inclusion of less accurate data from low flow stations. However the lack of any consistent trend in residuals and the good agreement (see Figure 22) between observed and predicted T_p confirms that the prediction equation is applicable to Nene ungauged catchments. Any minor timing differences in ungauged tributary hydrographs will become insignificant once the flows have been routed downstream to Wansford.

Table 6 Catchment Characteristics of Gauged Tributaries

Catchment	Area (km^2)	Length km	S1085 m/km	URBAN	% LAKE	SOIL	SAAR	RSMD
Nene at Upton	223.0	27.41	2.35	.006	0.06	0.45	668	24.26
Nene at St Andrews	232.8	20.86	4.18	.011	0.31	0.42	655	24.04
Ise Brook at Harrowden	194.0	38.94	2.15	.023	0.06	0.32	648	22.68
Harpers Brook at Old Mill	74.60	24.0	3.79	0.00	0.00	0.45	617	22.71
Willow Brook at Fotheringhay	89.62	33.98	2.87	.042	0.00	0.26	602	22.24

Table 7 Time to Peak Comparisons

	No of events	Time to peak, Tp (hours)		
		Mean Tp from observed flood events	Mean Tp estimated from observed LAG	Tp estimated from equation 6
Nene at Upton	7	11.2	13.4	14.8
Nene at St Andrews	4	13.7	16.1	11.4
Ise Brook	4	17.6	16.1	16.0
Harpers Brook	4	8.8	7.9	12.6
Willow Brook	5	18.0	17.7	13.7
Flore. Expt.	8	4.3	-	5.1
Grendon Brook	11	-	8.1	10.8
Southwick Brook	9	-	6.5	8.6
Wooton Brook	11	-	10.6	12.1
Wittering Brook	7	-	7.8	6.8

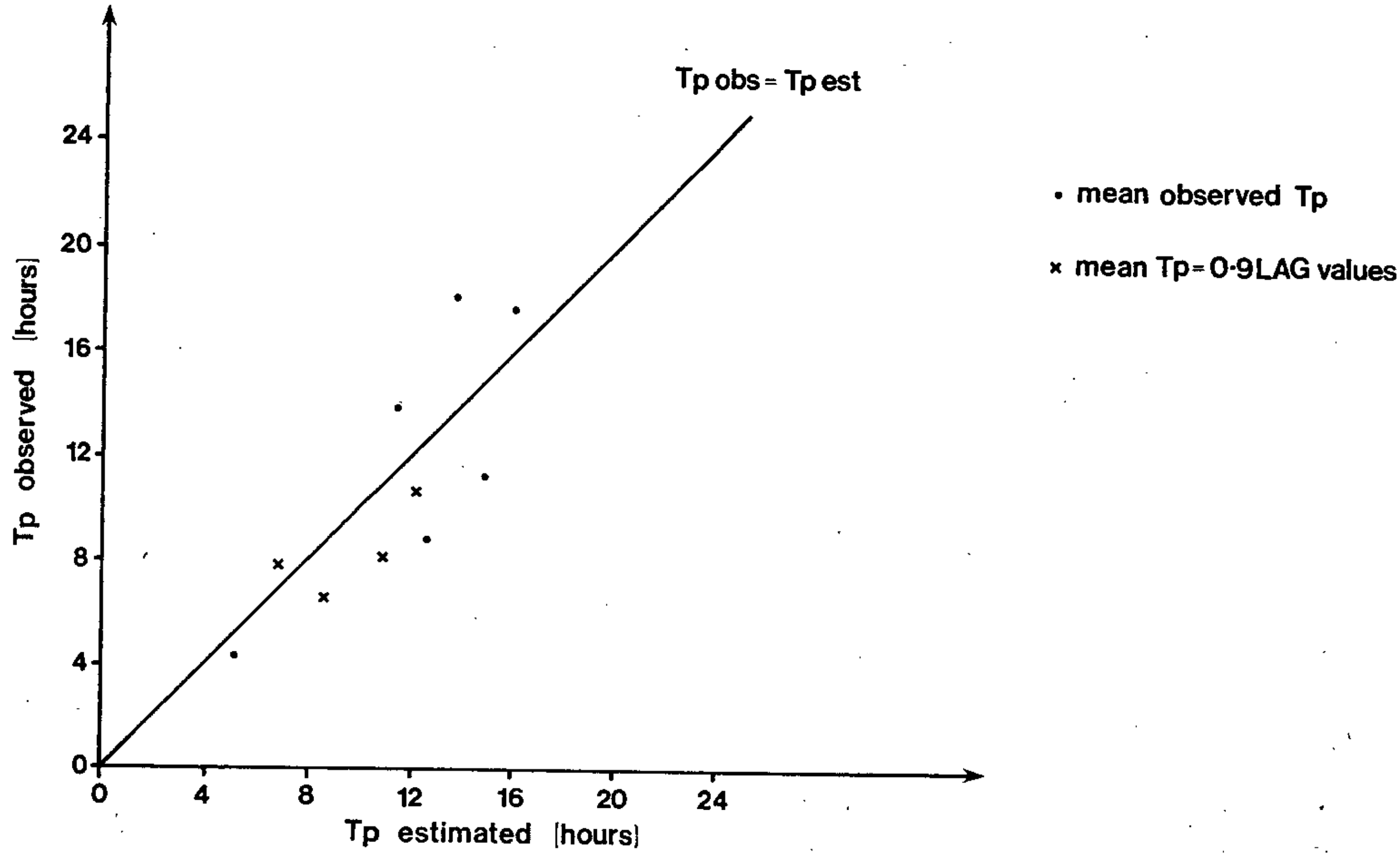


Figure 22. Comparison of observed and estimated time to peak values

6.0 HYDROLOGICAL INPUTS FOR A CALIBRATION EVENT

In order that the Nene river model may be calibrated and its predictive ability tested, hydrological inputs are required for an observed rainfall event. Recorded data will be input as far as possible, and the performance of the model assessed by comparing observed river levels at specified points along the Nene with those predicted by the model. Model parameters can then be adjusted where necessary, to improve the predictions.

6.1 Calibration event

This section outlines the procedure for calculating the input hydrographs from the tributaries for a calibration event.

(i) Gauged catchments

Recorded hydrographs for the River Nene at Upton and St Andrews, Ise Brook, Harpers Brook and Willow Brook should be input to the model. If data is unavailable, possibly through instrument failure, the flood hydrograph can be predicted by convoluting the average storm profile for the catchment with a unit hydrograph. This procedure is outlined in detail below for the ungauged tributaries. For the gauged areas a mean SPR from observed data (see Table 9) and an observed average unit hydrograph should be used. Hourly ordinate values of the unit hydrograph for the gauged catchments are given in Table 8.

(ii) Ungauged catchments

The ungauged catchments for which predicted flow hydrographs are required are outlined in Table 5. The steps in the calculation can be summarised as follows.

(a) Compute an average storm profile for each catchment of interest, using the method outlined in Vol IV, 3.2 of the Flood Studies Report. This involves calculating an average rainfall for each day of the storm using daily gauges on and in the vicinity of the catchment, and distributing this total in time according to the nearest autographic raingauge profile, or taking a distance weighted average if there are several gauges. As many of the catchments are small, the same rainfall profile may be applicable to neighbouring catchments. Storm depths could alternatively be obtained for each catchment by interpolation from isohyetal plots. It was decided

Table 8 Hourly ordinates of catchment average unit hydrographs (in cumecs/100 km²)

R. NENE AT UPTON MILL (32006)

.00	.48	1.37	3.21	4.24	6.39	7.96	9.91	10.60	11.42
12.06	12.58	12.23	11.99	11.19	10.56	9.63	9.00	8.27	7.46
7.04	6.40	5.96	5.53	4.96	4.77	4.33	4.02	3.79	3.49
3.35	3.26	3.07	3.04	2.93	2.85	2.77	2.68	2.61	2.52
2.39	2.27	2.19	2.11	2.00	1.90	1.76	1.70	1.60	1.50
1.42	1.33	1.30	1.22	1.16	1.10	1.02	.93	.83	.78
.71	.70	.65	.61	.57	.52	.48	.44	.41	.40
.37	.35	.34	.31	.31	.29	.28	.25	.25	.24
.24	.23	.20	.18	.00					

R.NENE AT ST ANDREWS (32007)

.00	.50	1.60	2.00	3.20	4.72	6.87	7.98	9.37	10.27
10.65	11.40	11.58	12.16	12.64	12.71	12.43	11.44	10.24	9.14
8.39	7.91	7.40	7.00	6.57	6.12	5.61	5.12	4.68	4.42
3.94	3.54	3.08	2.80	2.59	2.38	2.15	2.00	1.88	1.77
1.62	1.55	1.53	1.51	1.48	1.41	1.34	1.29	1.20	1.12
1.05	.97	.88	.80	.70	.62	.55	.49	.42	.38
.34	.30	.25	.25	.24	.23	.23	.22	.21	.21
.19	.18	.16	.15	.14	.11	.10	.09	.08	.00

ISE BROOK AT HARROWDEN (32004)

.00	1.20	2.22	3.15	3.95	4.68	5.37	5.94	6.42	6.75
6.92	7.00	7.02	7.00	6.95	6.85	6.75	6.70	6.71	6.82
6.95	7.12	7.18	7.15	7.05	6.87	6.60	6.25	5.90	5.55
5.25	5.01	4.80	4.58	4.37	4.16	3.95	3.75	3.57	3.40
3.22	3.05	2.87	2.70	2.50	2.35	2.20	2.07	1.95	1.85
1.75	1.65	1.57	1.50	1.40	1.30	1.25	1.20	1.15	1.10
1.07	1.00	.95	.91	.88	.83	.79	.72	.68	.66
.61	.57	.54	.48	.44	.42	.38	.37	.35	.34
.33	.31	.30	.29	.27	.26	.25	.24	.23	.20
.19	.18	.18	.17	.00					

HARPERS BROOK AT OLD MILL BRIDGE (32003)

.00	1.42	3.47	6.79	9.59	12.30	14.99	17.90	20.02	21.86
22.45	22.55	20.07	14.07	10.39	7.42	5.71	5.30	4.91	4.50
4.11	3.82	3.51	3.28	2.99	2.71	2.49	2.31	2.14	1.97
1.84	1.79	1.66	1.47	1.30	1.11	.91	.68	.49	.32
.18	.07	.00							

WILLOW BROOK AT FOTHERINGHAY (32002)

.00	.66	1.25	1.90	2.55	3.22	3.93	4.57	5.12	5.72
6.30	6.80	7.40	7.97	8.48	8.97	9.20	9.35	9.40	9.35
9.15	8.90	8.57	8.27	8.00	7.72	7.40	7.07	6.65	6.12
5.60	5.07	4.70	4.30	3.96	2.50	2.35	2.20	2.10	1.97
1.85	1.75	1.67	1.55	1.45	1.37	1.30	1.25	1.15	1.10
1.02	.97	.89	.83	.76	.72	.65	.60	.57	.49
.48	.41	.37	.34	.30	.28	.22	.20	.18	.00

Table 9 Input parameters for design and calibration

Catchment number	Area (km²)	T_p (hours)	SPR(%)	Design CWI (mms)	SBF (cumecs/km²)
Gauged					
32007	232.8	Use catchment average UH's (Table 8)	26	98	.0209
32006	223.0		26	99	.0208
32004	194.0		32	97	.0198
32003	74.6		41	88	.0198
32002	89.6		23	85	.0195
Ungauged					
1	110.9	14.6	37	91	.0193
2	66.4	8.1	30	86	.0192
3	15.8	8.1	31	84	.0171
4	29.8	8.5	29	84	.0201
5	31.0	6.4	37	85	.0199
6	26.9	8.1	39	85	.0198
7	39.4	8.2	47	85	.0198
8	20.8	8.0	48	85	.0206
9	40.4	8.5	45	82	.0189
10	24.2	7.8	12	81	.0206
11	40.2	6.7	9	93	.0195
12	31.4	8.3	25	88	.0199
13	22.5	8.8	24	90	.0204
14	21.1	8.3	42	88	.0207
15	28.5	8.3	42	85	.0205
16	43.9	10.3	15	86	.0203
17	36.0	5.9	22	85	.0193
18	12.1	6.1	33	87	.0213
19	30.2	6.1	35	87	.0207
20	16.0	6.8	29	88	.0212
21	20.8	8.1	24	88	.0212
22	18.8	2.5	38	87	.0207
23	16.8	4.1	42	87	.0207

to standardize on a data interval of 1 hour for all catchments. This may produce a slightly less well defined hydrograph for the two flashiest catchments (22 and 23), but overall the simplification will have a negligible effect.

(b) Compute an observed CWI for each catchment, based on the soil moisture deficit from the nearest SMD station and API5, based on the daily raingauge nearest to the centre of the catchment.

(c) Convolute the rainfall profile with the synthetic triangular one hour unit hydrograph and add a baseflow component to give the predicted flood hydrograph. A listing of the Fortran computer program to do this is given in Appendix B. Baseflow is input in terms of standard baseflow (SBF) estimated from

$$\text{SBF} = 0.00074 \text{ RSMD} + 0.003$$

which is converted in the program to total baseflow as follows

$$\text{Baseflow} = (0.00033\ (\text{CW1}-125) + \text{SBF})\ \text{AREA}.$$

Inputs to the program are thus hourly rainfall ordinates, the appropriate SPR, SBF and T_p values from Table 9 and the calculated CWI value. For those catchments with low flow stations (2,10), a mean T_p value based on LAG has been included.

6.2 Input points along the Nene

As indicated in Figure 16, 20 points along the River Nene have been specified as suitable input points for the observed and predicted hydrographs. Where possible the input points were chosen to coincide with the confluence of a tributary with the main river and in several cases, more than one hydrograph is input at a particular point. In view of the marked attenuation of hydrographs along the Nene, the effects of these small simplifications will be negligible. The catchment hydrographs to be input at each of the 20 nodes are given in Table 10.

For several catchments, as indicated in Table 10, it is more realistic to distribute runoff along a reach of the Nene, rather than concentrating inflow at one point. These are generally catchments immediately adjacent to the Nene with few significant drainage channels. The predicted hydrograph for the catchment is distributed equally (by scaling ordinates) to a number of additional lateral inflow points along the reach, as specified by the River Division.

Table 10 Input points for Nene model

		Contributing catchments	
Input point	Grid reference	Gauged	Ungauged
1	SP 721 590	32006	1
2	SP 754 597	32007	22,23
3	SP 810 610	-	21
4	SP 845 617	-	20
5	SP 887 645	-	2
6	SP 897 660	-	3
7	SP 907 670	32004	19,18,17 distributed
8	SP 925 680	-	4,17 distributed
9	SP 957 705	-	5,17 distributed
10	SP 965 740	-	6,17 distributed
11	SP 992 780	-	16
12	SP 997 802	32003	-
13	TL 020 815	-	7,15 distributed
14	TL 037 877	-	14,15 distributed
15	TL 050 867	-	8,15 distributed
16	TL 040 900	-	13
17	TL 045 925	-	12
18	TL 075 922	-	9
19	TL 075 935	32002	11 distributed
20	TL 085 970	-	10,11 distributed

7.0 HYDROLOGICAL INPUTS FOR A DESIGN EVENT

The main objective of this part of the study is to specify a design storm input, which will result in a flood of the required return period over the Nene catchment. It is also important to be able to check the frequency or return period of predicted floods at as many sites as possible. The analyses directed towards this secondary aim will be described first. Clearly different features of a flood will be important depending on the design situation - either peak levels, peak flows or volumes of runoff. Flood frequency relationships can be derived based on all three types of data, and the analysis of peak flow and volume data is outlined here. An identical procedure could be adopted for the level recording stations on the Nene at Northampton, Wollaston and Lilford respectively (see Figure 2).

7.1 Flood frequency study

Flow records were statistically analysed for each of the five gauged tributaries and the Nene at Wansford. The annual maximum values listed in Table 11 were extracted from the records as outlined in Section 3.3, and a general extreme value distribution was fitted to the data (see FSR 1, 2.3). As indicated in Figures 23 to 27 values of $Q(T)/\bar{Q}$ were plotted against reduced normal variate y_i (and hence return period) and an eye guided line drawn through the points. Division of all flows by the mean annual flood ($\bar{Q}$) effectively standardised the data and enabled comparison of growth curves both between catchments and with the region curves for areas 4 and 5 (see Figure 28). The East Anglian region curve was derived by combining data from all stations in hydrometric areas 30 to 35. The region 5 growth curve was also plotted since the Nene is close to the boundary with that region. As expected with only five stations there was some variability, but the general trend of the region curves was confirmed.

It was decided to combine these curves to produce a Nene region curve. This would be generally applicable to catchments in the Nene area, and as it is based on more information was felt to be an improvement on the individual curves. Also the Nene area is not entirely typical of either E. Anglia or Severn Trent, and is probably better represented by a curve between these two. Consequently it is recommended that use is made of the Figure 28 curve labelled 'Nene region curve' for the purposes of this study.

Table 11 Flood Frequency Data

Annual Maximum Discharges (cumecs)

Rank	Nene at Upton 32006	Nene at St Andrews 32007	Ise Brook at Harrowden 32004	Harpers Brook at Old Mill 32003	Willow Brook at Fotheringhay 32002
1	19.60	31.51	30.03	18.95	17.00 (E)
2	19.60	26.49	28.39	18.58 (E)	15.60
3	19.14	25.60	26.32	17.42	15.00
4	19.14	25.28	24.04	17.39 (E)	8.74
5	19.06	24.80	23.73	17.02 (E)	8.37
6	17.98	24.68	19.31	16.08 (E)	7.79
7	17.50	22.90	18.25	14.72	7.79
8	17.19	22.46	18.22	12.78	7.73
9	16.03	22.31	17.46	12.56	7.53
10	14.84	22.14	16.90	11.27	7.48
11	2.94	21.26	16.74	10.90	7.26
12		21.25	16.73	10.53	6.94
13		20.46	16.61	10.52	6.88
14		20.33	16.61	9.74	6.51
15		18.17	16.56	7.90	6.34
16		18.17	16.37	7.68	6.26
17		17.56	15.91	7.32	5.78
18		17.56	15.77	7.07	5.52
19		14.54	15.18	7.02	5.35
20		14.01	14.56	6.90	5.21
21		14.01	14.12	6.83	5.02
22		13.33	14.01	6.29	5.01
23		12.97	13.90	5.85	4.93
24		12.30	13.11	4.85	4.67
25		12.15	12.32	4.72	4.53
26		12.01	10.96	4.69	4.25
27		11.95	10.77	3.90	4.16
28		11.08	10.18	3.75	3.51
29		10.90	9.85	3.63	2.77
30		10.21	9.53	3.53	2.75
31		10.10	8.62	3.45	2.72
32		9.46	7.26	2.97	2.69
33		9.18	6.21	2.63	2.55
34		8.96	4.66	2.63	2.55
35		6.00	4.00 (E)	2.38	2.41
36		5.67		1.22	2.41
37		4.05		1.16	2.38
38		3.47		0.71	1.42
39					1.22
40					0.74

E - estimate

Figure 23. Flood frequency curve-River Nene at Upton Mill

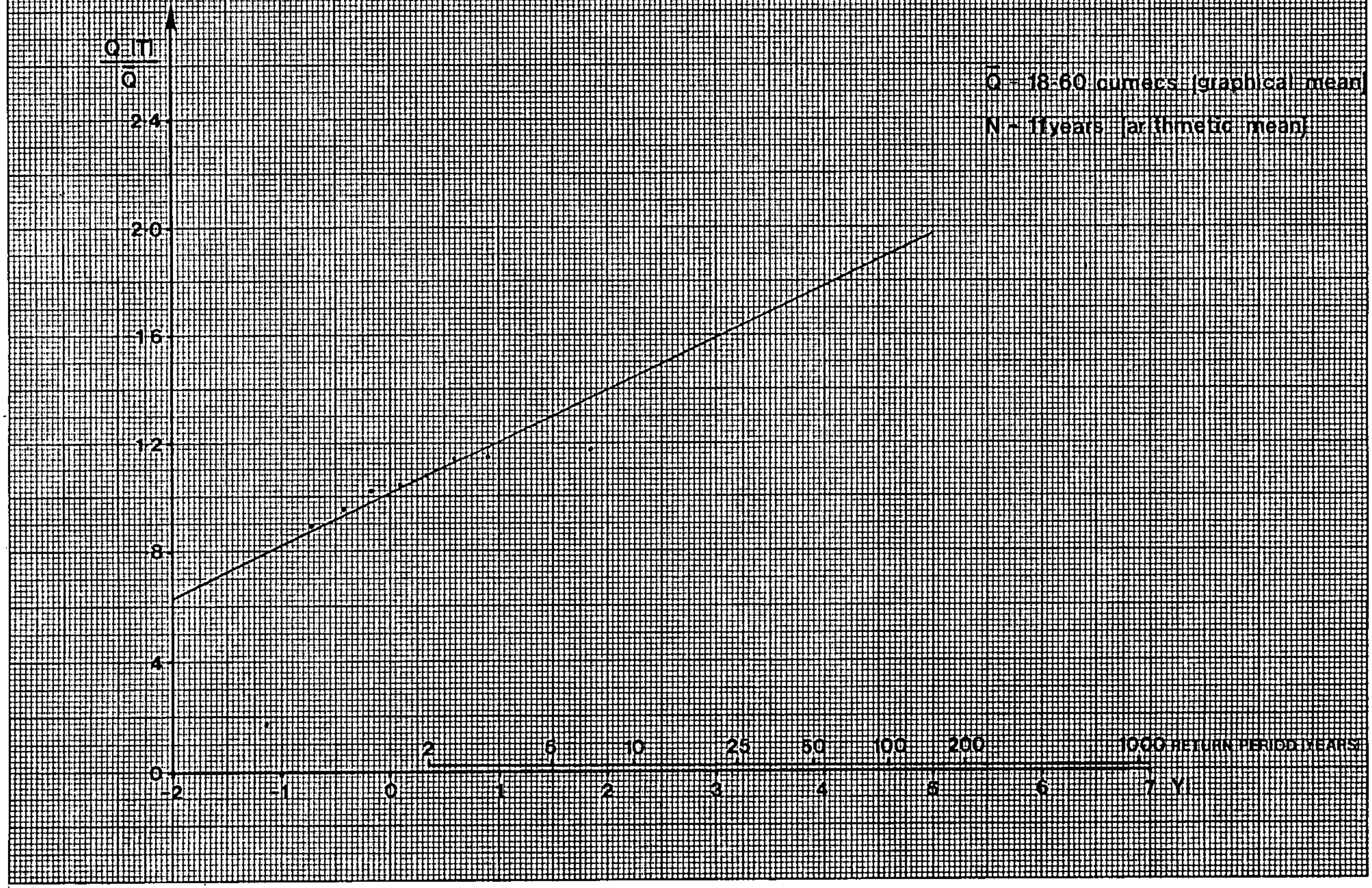


Figure 24. Flood frequency curve - River Nene at St. Andrews

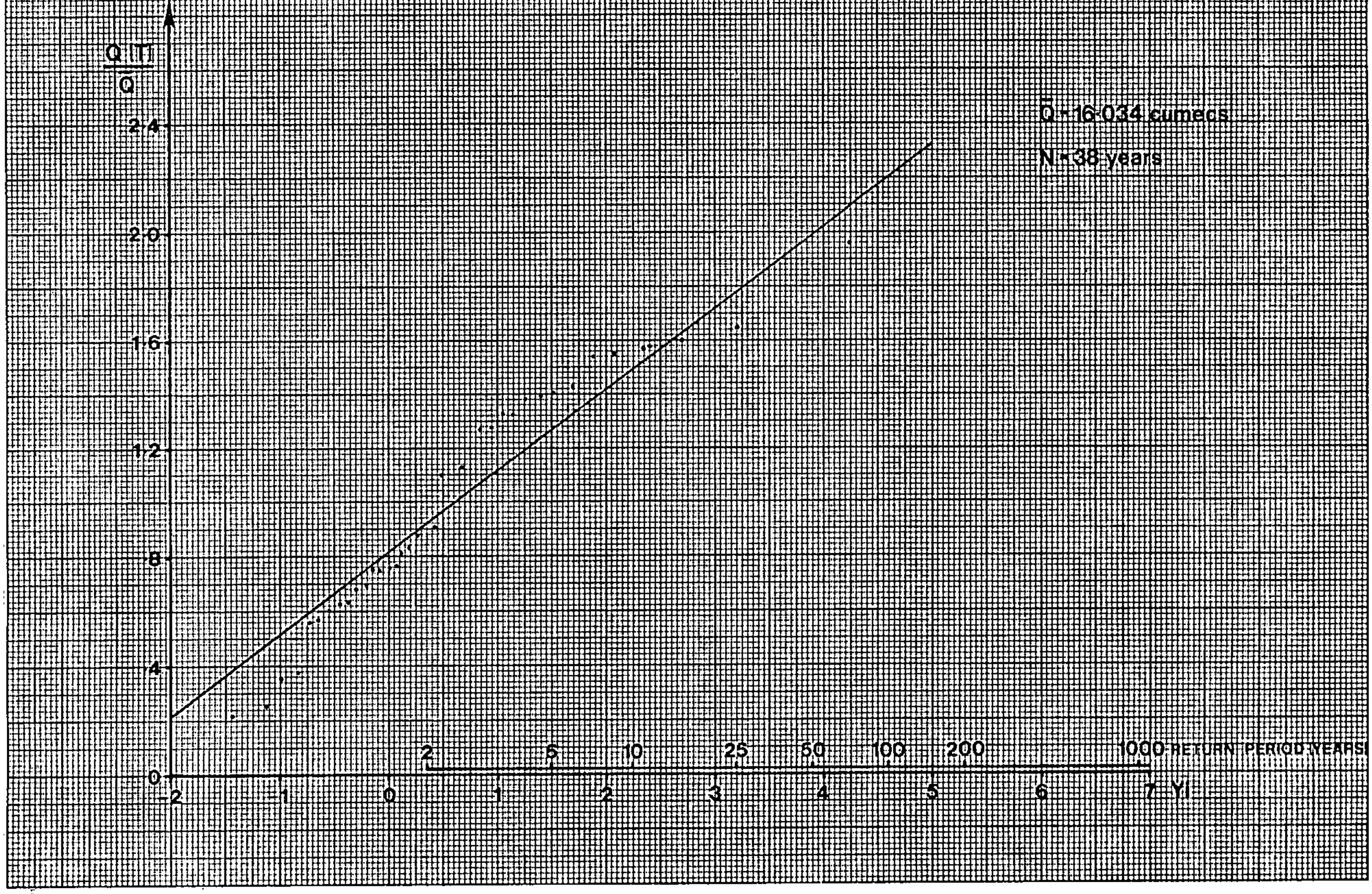
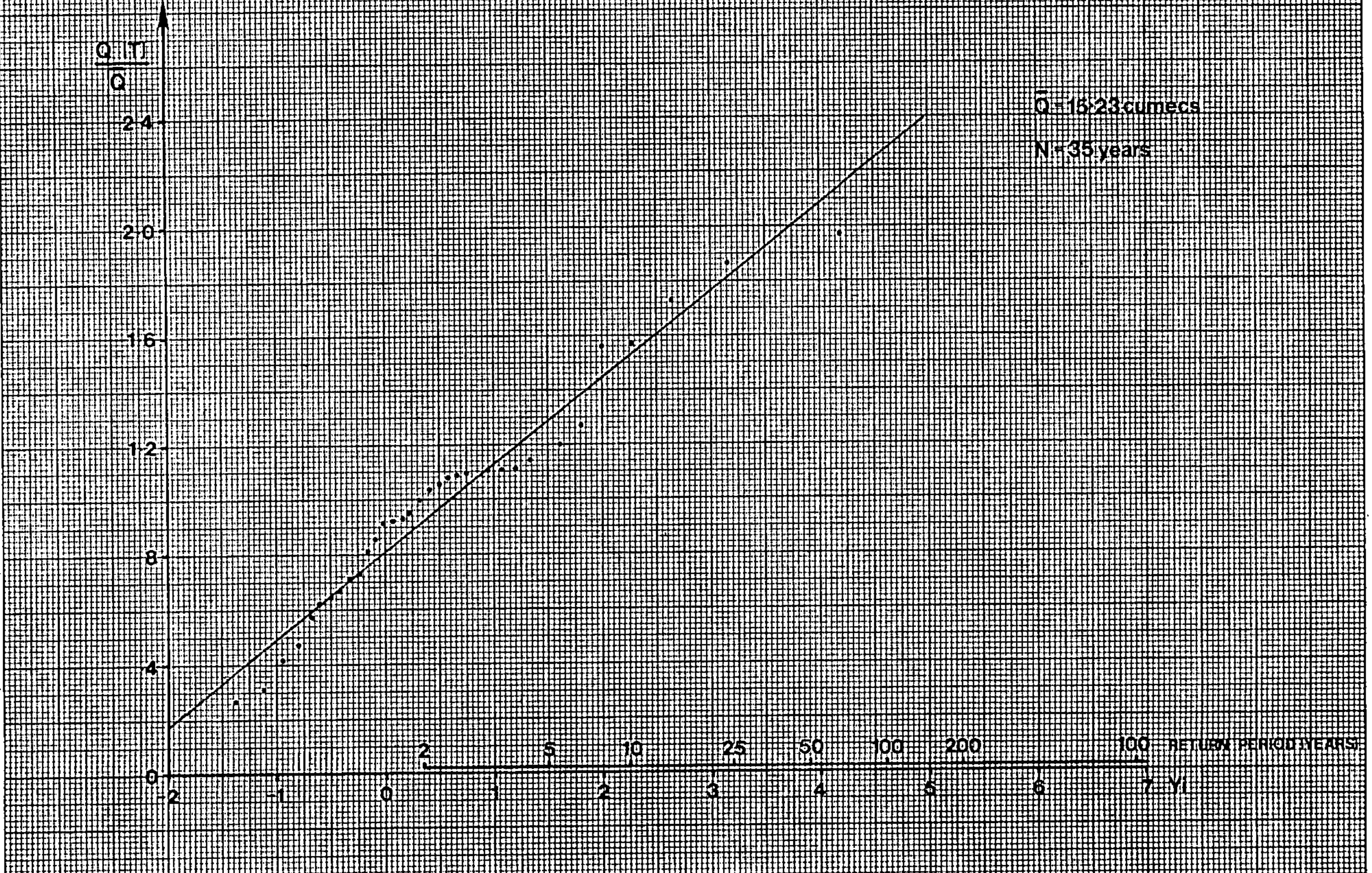


Figure 25. Flood frequency curve - Ise Brook at Harrowden



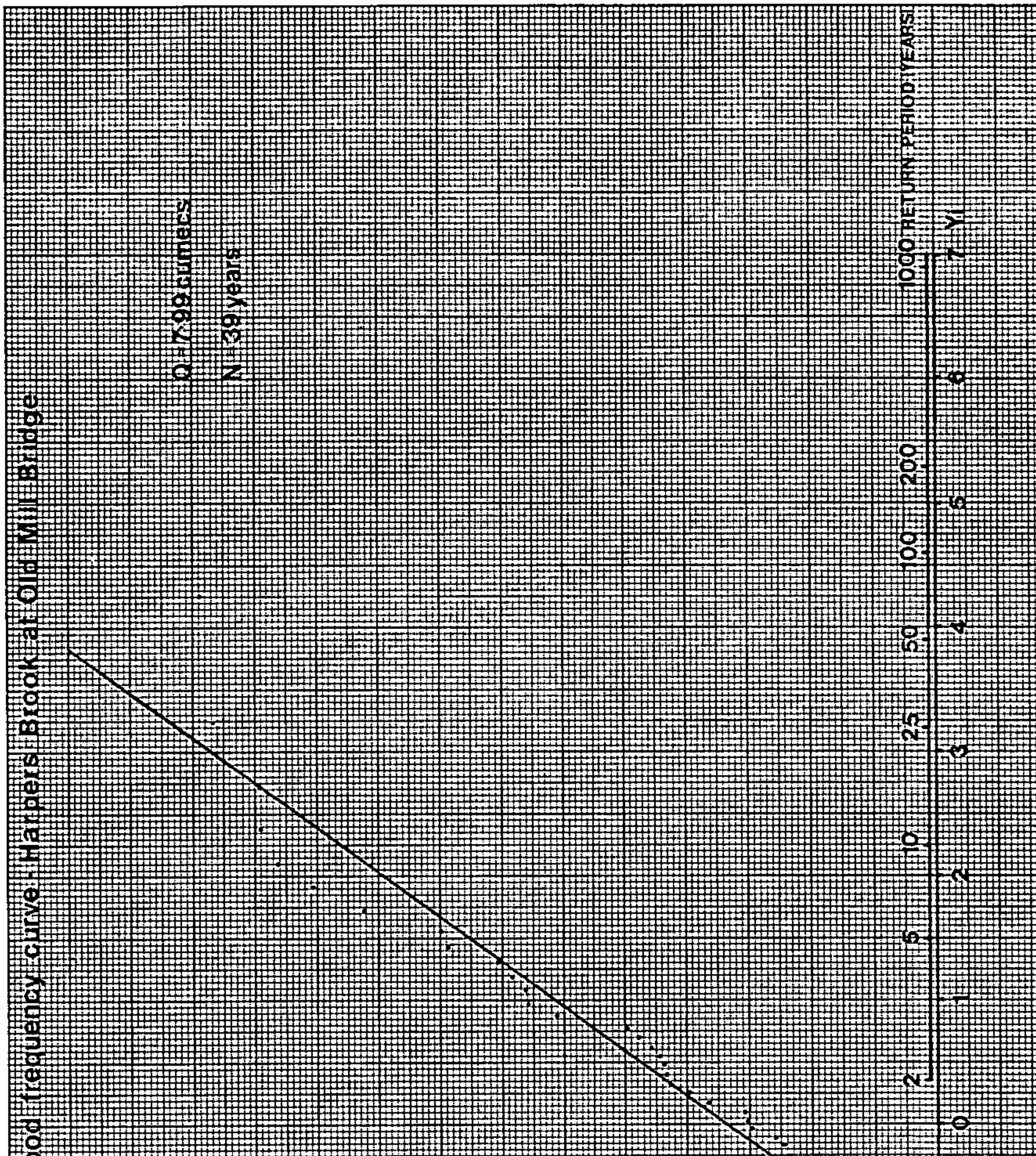
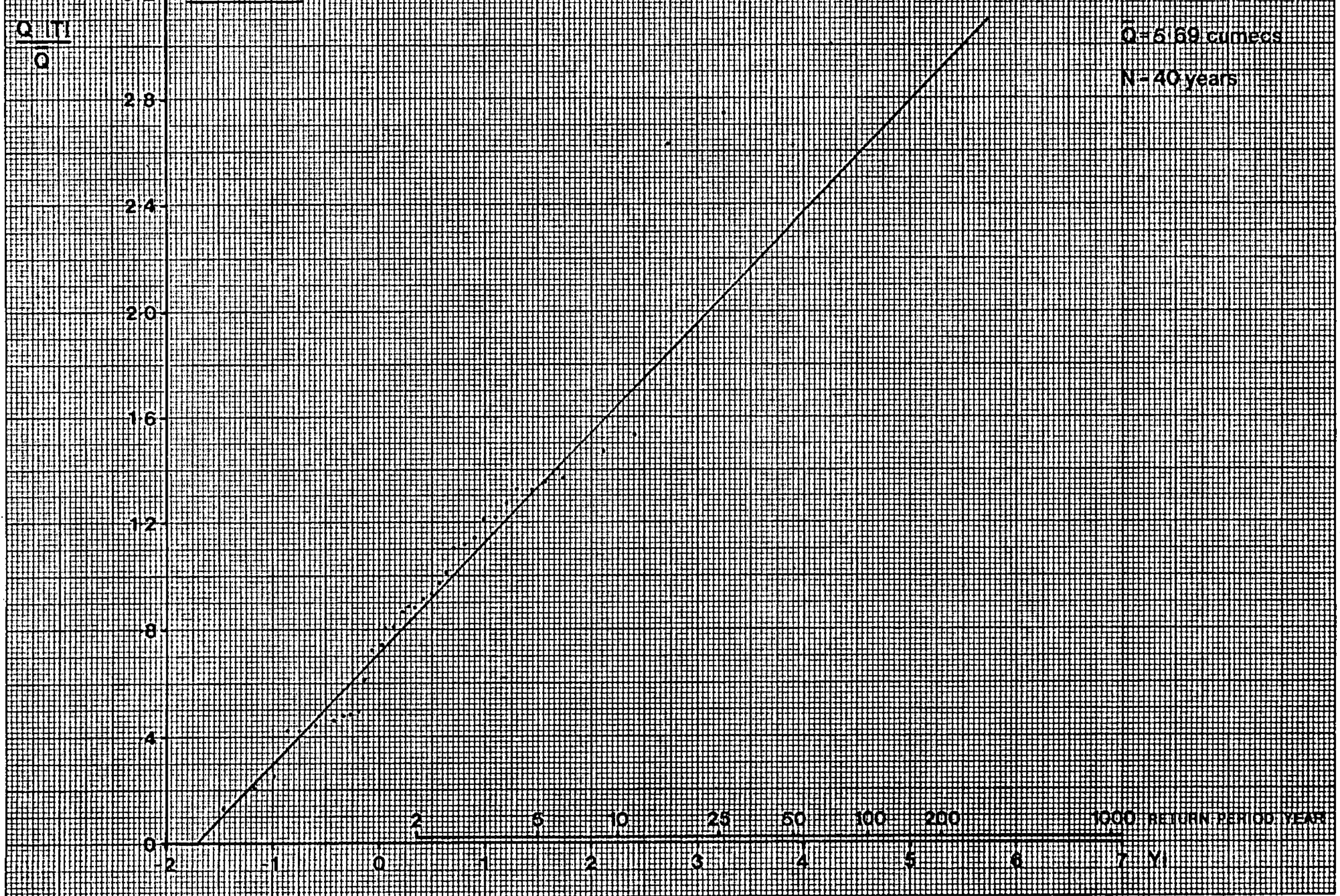


Figure 26 Flood frequency curve - Harpers Brook at Old Mill Bridge

Figure 27. Flood frequency curve - Willow Brook at Fotheringhay



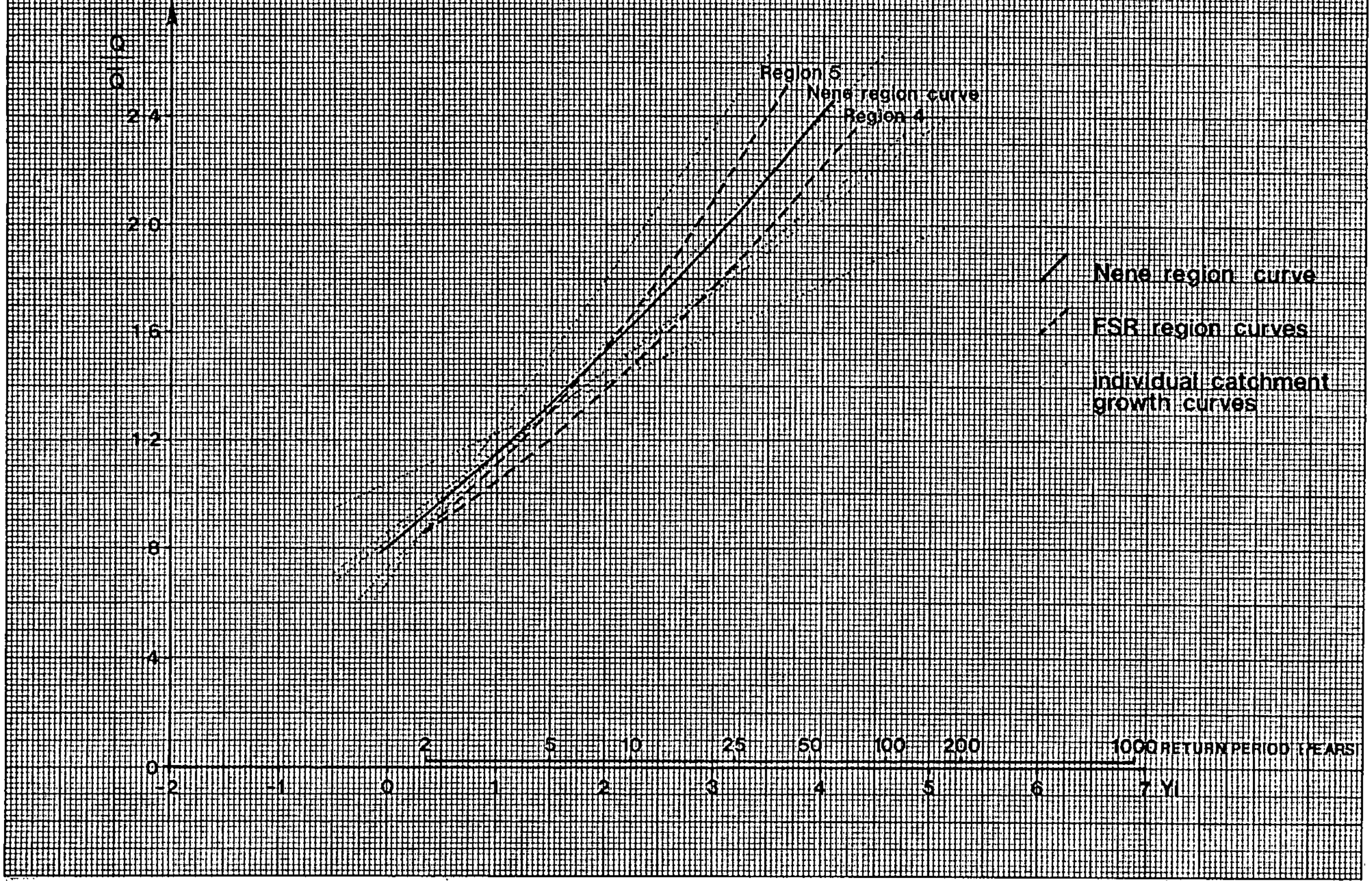


Figure 28. Growth curves

7.2 Volume flood growth curves

As noted earlier the flood peak at Wansford is sustained for 2 to 3 days. Thus the duration of flows of a particular size or the volume of runoff is of critical importance. Volume flood growth curves were derived using a similar procedure to peak flow data. The annual maximum flows averaged over different durations (namely 1,2,5,10,15 and 20 days) were extracted by computer program from the mean daily flow records for the Nene at Wansford (see Table 12). The mean daily discharges were calculated using low flow data from Orton and high flow data from Wansford. Data were extracted by passing a moving average filter of the required duration through the record, and calculating the maximum average flow for each duration in each year.

Flood frequency curves were derived for each duration individually, using the same method as for instantaneous flood peaks; they are plotted together with instantaneous growth curve in Figure 29. The close agreement between maximum instantaneous and daily flows is to be expected, considering the non peaky nature of the hydrographs at Wansford.

7.3 Design storm

Ideally a design storm profile is required which will result in a flood hydrograph of a realistic shape and whose peak flow, volume relationships correspond to a specified return period along the main part of the River Nene. Clearly a design storm which can meet all these criteria is unattainable. A single design storm for the whole Nene catchment is preferred, yet with such a large and diverse area, it could result in floods of differing return periods for each tributary. However to derive different design storms for each catchment, in order to maintain a constant return period would be both unrealistic and computationally unmanageable. Also the resulting flood at the downstream end of the Nene would probably be of a much larger return period than required.

It was therefore decided to compute a design storm profile which would result in a flood of the required return period (in this case 10 years) for the mid reaches of the Nene. The first and probably most important step in defining the storm profile, is determining a suitable design storm duration. The Flood Studies catchment characteristics method is unsuitable for catchments

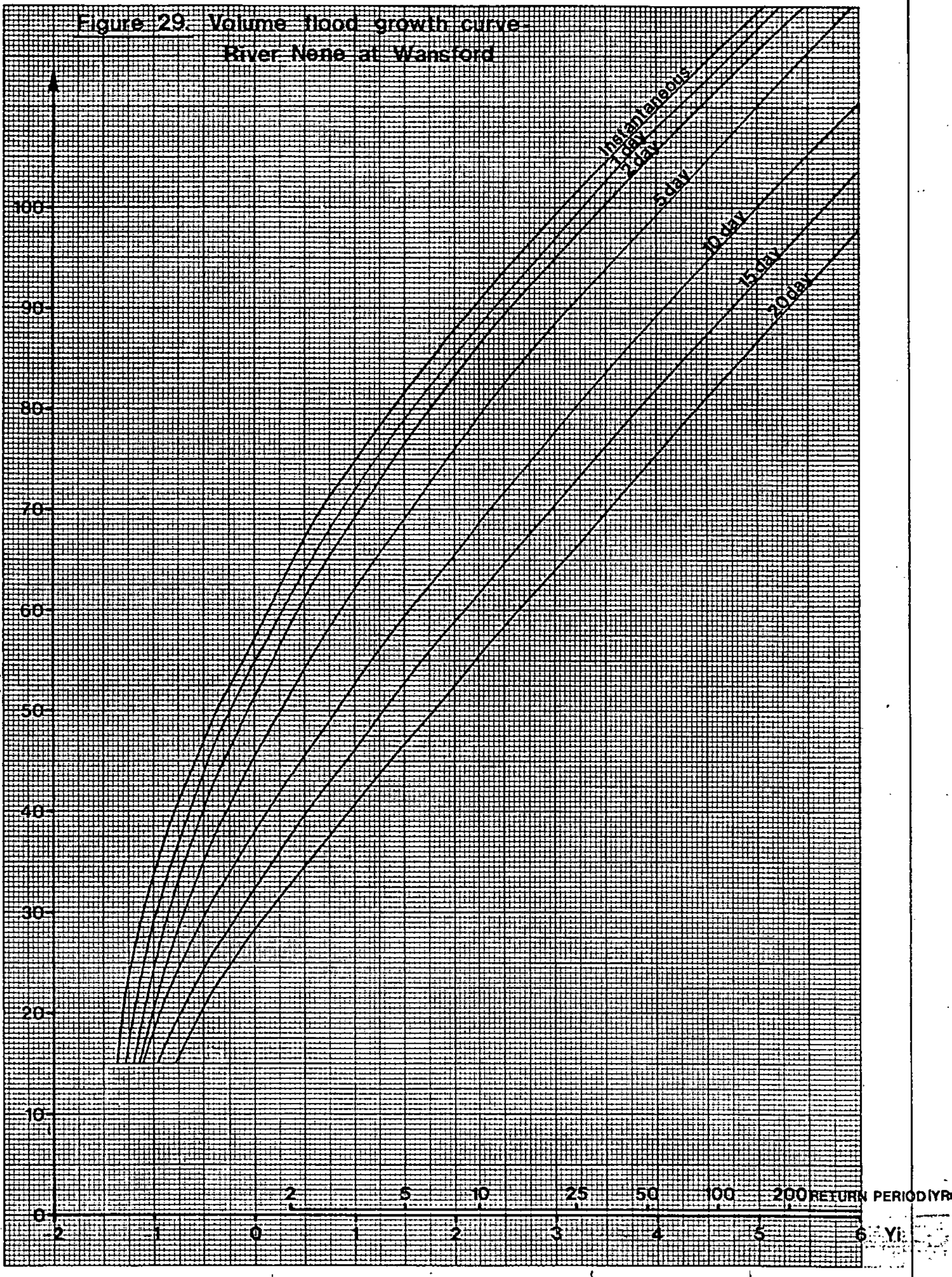


Figure 29 Volume flood growth curve-
River Nene at Wansford
Instantaneous
1 day
2 day
5 day
10 day
15 day
20 day
Q̄ [D] cumecs
0
10
20
30
40
50
60
70
80
90
100
2
5
10
25
50
100
200
RETURN PERIOD(YRS)
-2
-1
0
1
2
3
4
5
6
Yi

Table 12 Average annual maximum flows (in cumecs) for different durations at R. Nene at Wansford

1DAY		2DAY		3DAY		5DAY		10DAY		15DAY		20DAY	
18031947	319.98	17031947	278.07	17031947	260.61	15031947	238.71	14031947	176.05	12031947	139.73	11031947	116.18
23011959	98.54	22011959	96.99	21011959	94.77	21011959	90.44	23011941	73.57	28111954	62.12	28111954	65.22
29111954	92.60	29111954	90.61	28111954	88.73	25011941	83.37	18011959	71.76	22011941	61.77	7011959	60.46
10031941	90.05	27011941	85.38	26011941	84.38	27111954	82.18	8121954	70.74	9011959	56.07	23011941	55.36
18051967	86.65	18051967	84.95	12031975	82.12	12031975	77.23	9031975	66.55	18021977	55.41	10021977	52.13
13031975	85.86	13031975	83.15	17051967	80.14	15051967	75.44	20021977	64.34	22101960	54.22	21101960	47.11
16031969	85.42	3111960	80.56	2111960	79.00	24021977	72.99	27101960	62.16	9031975	53.29	9031975	46.28
3111960	83.53	15031969	78.91	14031969	76.69	1111960	72.83	15011969	58.91	13011969	52.30	9121965	45.34
27021977	81.24	26021977	77.06	25021977	75.01	13031969	71.73	24011971	57.16	21011971	50.78	12011969	45.15
3021943	78.15	3021943	73.34	27021958	68.43	25021958	65.53	18121965	56.38	13121965	47.88	21011971	43.59
28021958	73.34	27021958	70.37	14031970	67.96	25011971	63.63	13051967	53.53	25011942	43.85	25011942	41.75
8011951	71.64	27011971	69.29	27011971	67.68	13031970	63.27	23021958	51.97	14031951	43.10	11021958	40.71
27011971	70.84	8011951	68.81	3021943	66.64	29011960	61.96	25011960	51.93	4021950	42.32	4021950	38.19
16031970	68.46	15031970	68.30	31011960	65.60	12071968	61.05	5011951	50.37	23011960	42.05	10031951	38.17
12071968	66.60	1021960	65.59	8011951	64.19	19121965	59.47	10071968	48.95	13051967	41.35	13051967	35.75
2021960	65.69	15071968	62.82	20121965	62.20	7011951	58.90	7021950	48.20	23021958	41.22	20011960	35.37
21121965	62.86	21121965	62.72	14071968	60.13	1021943	58.67	11031970	47.00	14031964	40.08	1021945	34.36
27011942	62.58	16031964	60.46	13021950	57.96	12021950	54.31	16031964	44.00	11031970	37.96	24011943	32.88
16031964	61.45	27011942	59.75	16031964	55.69	16031964	51.25	31011943	43.74	1021945	37.61	28021970	32.71
14021950	58.62	13021950	58.47	26011942	55.41	25011942	49.27	26011942	41.99	31011943	37.46	14031944	32.19
4021945	53.80	30031963	51.54	30031963	51.54	19121952	48.88	17121952	38.22	10071968	36.20	17121952	29.19
30031963	52.67	21121952	51.25	21121952	50.97	1021945	45.76	1021945	37.54	5031963	31.93	10071968	28.84
22121952	51.54	3021945	50.83	2021945	48.33	30031963	43.47	31121948	36.09	18121952	31.79	4031963	27.81
12031952	50.65	11031952	48.28	10031952	46.91	2011949	42.59	5031963	35.98	30121961	30.23	29121961	27.07
3011945	47.57	31011956	46.44	2011949	45.40	29011956	42.59	28121956	34.92	26011956	29.93	23011956	26.61
1021954	46.72	2011949	46.02	31011956	45.12	9031952	42.31	26011956	34.51	25121956	28.71	6021957	25.10
8031972	46.45	7031972	45.51	6031972	44.58	29121956	41.80	5011962	34.50	31121948	28.49	29121951	24.14
12011962	45.59	31121956	44.46	10021974	43.96	9011962	40.32	9021974	32.54	6021974	27.27	31121948	23.80
31121956	44.74	10021974	44.09	30121956	43.70	9021974	39.66	7031952	30.17	31011946	26.90	30011946	23.39
10021974	44.43	12011962	43.89	11011962	43.23	5031972	38.51	3021946	29.53	29121951	26.18	6021974	23.29
28061973	41.64	28061973	39.29	28061973	38.61	27061973	34.79	5031972	29.42	2121972	25.42	27011972	23.10
20021954	34.55	20021954	34.41	8021946	33.23	6021946	31.77	2121972	27.94	27011972	23.95	2121972	21.93
9021946	34.26	8021946	33.56	14061954	30.80	19021954	25.05	14021954	19.23	10021954	18.51	10021954	16.03
22031965	31.71	22031965	30.72	22031965	26.73	22031965	23.68	20031965	18.90	21031965	14.26	13011948	13.03
31011948	25.91	30011948	25.10	30011948	24.65	30011948	21.45	26011948	16.47	26011948	14.06	20031965	11.64
5041944	13.22	5041944	12.86	4041944	11.69	4041944	9.46	4041944	6.41	19111975	5.72	17111975	5.56
3121975	12.93	2121975	9.97	2121975	8.35	30111975	6.96	27111975	6.10	3041944	4.96	4041944	4.18

this size, since they were not represented in the original data set. As shown in Figure 2, the Nene is gauged for levels only, between Northampton and Wansford, at Wollaston and Lilford respectively. These enable the time of peak level, or in the case of multipeaked events the centroid of peak levels to be determined for a range of events. If the centroid of the corresponding rainfall profile for the catchment upstream of these points is also computed, the difference in time between the centroids of rainfall and peak level gives the catchment lag. Hence T_p can be calculated and from this the design storm duration D (in hours), as follows:

$$D = T_p (1 + SAAR/1000) \qquad (10)$$

SAAR is the standard average rainfall for the catchment (mms). The lag to Wansford was calculated in a similar manner, based on peak flows rather than levels.

The mean catchment lags computed for Wollaston, Lilford and Wansford were 35, 45 and 50 hours respectively which, using equation (10), result in storm durations of 51, 65 and 72 hours. A design storm based on the Nene at Lilford was felt to be most realistic for the mid reaches of the Nene. The storm depth corresponding to a duration of 65 hours, and a 10 year return period flood, was then calculated using the method outlined in Vol I, 6.8.2 of the Flood Studies Report and distributed in time according to the 75% winter profile.

This design storm, calculated solely by Flood Studies methods, though improved by incorporation of local data, still has certain serious drawbacks. A design storm duration based on Lilford is clearly an unrealistic over-estimate for the small tributary catchments, and will result in individual catchment floods of return period less than 10 years. Once tributary inflows combine the return period will be increased, but from the hydrology alone it is impossible to say by how much. Analyses of historical peaks at Wansford and corresponding peaks on the five gauged tributaries revealed no consistent trend in terms of their return period, and 10 year floods at Wansford could be produced from 10 ± 5 year floods on the tributaries (see Table 13). Secondly once runoff enters the Nene, the hydrographs are influenced by storage and routing effects which can only be simulated using the mathematical model.

Table 13. Comparison of ranking of flood peaks at Wansford and Nene tributaries, resulting from the same rainfall event

Event Date	Nene at Wansford	Nene at St Andrews	Ise Brook	Harpers Brook	Willow Brook
	Rank in annual maximum series of each catchment*				
	(N = 41)	(N = 38)	(N = 35)	(N = 39)	(N = 40)
18/3/47	1	5	2	1	3
9/2/40	2	-	-	9	18
12/3/75	3	3	6	5	1
9/1/59	4	2	4	8	-
27/2/77	5	32	10	-	8
10/3/41	6	1	-	10	13
15/5/67	8	8	7	13	5
16/3/69	10	-	-	7	4

* 1 = largest peak flow in annual maximum series of length N years.

In view of these problems it is recommended that hydrological inputs to the model be calculated for a range of different design storms and run through the Nene mathematical model.The return period of predicted levels at points along the Nene can be checked, and the design storm chosen which most closely reproduces 1 in 10 year levels along the mid reaches of the Nene. Welland and Nene River Division have undertaken to derive level frequency relationships for Lilford and Wollaston, to enable these checks to be made. Additional checks can be made of the return period of peak flows and critical volumes (for 2 or 3 day durations) at Wansford, using Figure 29.

It is recommended that the storm depth (60 mm) and duration (65 hours) calculated for Lilford be used as a starting point. Other suggested combinations of depth and duration are shown in Table 14; these encompass the type of storms observed over the Nene area.

Although it may be possible to reproduce a flood of a specified return period at a point, a consistent return period is unlikely to be maintained over a long reach. It is felt that flows will be sufficiently consistent, if return periods (T) are in the range $5 < T < 20$ years. If this cannot be attained then it may be necessary to derive separate design storms for the upper and lower reaches of the Nene, and the values in Table 14 might serve this purpose. These simulation exercises should however give a 'feel' for the catchment response of the Nene, and may suggest other improvements to the design storm input.

7.4 Washland storage area

The analyses, outlined in this report, have been based on historical flood data prior to 1980; however the applications of the Nene model must consider a washland storage area recently constructed downstream of Northampton. This is an off channel storage area of capacity 2.4 million M^3, designed to store excess runoff created by the urban development of Northampton. By cutting off flows in the main channel above 25 cumecs, the same volume of runoff is maintained in the Nene, as before urban development took place, although the flood peak will be severely truncated. The reservoir has been in operation since summer 1980.

There is clearly insufficient recorded data to assess the effect of the washland area on levels and flows downstream. Its effect can best be predicted by a simulation type exercise using the Nene model to predict downstream levels both with and without the storage reservoir. If the simulations are repeated for a range of storms, a 'pre versus post' reservoir level relationship can be built up for the Nene at Wollaston and Lilford. Similarly the effects on the flood hydrograph at Wansford can be established though it is anticipated that these will be minimal. The level or flood frequency relationships at these sites can then be adjusted to take account of this additional storage area.

Table 14. Suggested design storm durations and depths for simulation

Duration (hours)	Depth (mm)
75	70
65	60
55	50
45	

7.5 Hydrological inputs

The procedure for computing input hydrographs is essentially the same in design as calibration. In design the input data required is as follows:

(i) Decide on a single design storm profile (in initial stages at least) which can be applied to all catchments. This is input as hourly rainfall ordinates.

(ii) For gauged tributaries, use the average unit hydrograph derived from observed data (ordinates given in Table 8).
For ungauged catchments input the relevant T_p value, given in Table 9.

(iii) SPR, CWI, and SBF values for all catchments are given in Table 9 for the design case.

8.0 CONCLUSION

This report has described the work carried out by the Institute of Hydrology to provide flow inputs to a mathematical model of the River Nene. Figure 16 shows the 20 input locations. Use was made of the extensive flow and rainfall records to calibrate the FSR Unit Hydrograph/Losses Model from observed flood events on five gauged catchments. For ungauged areas the time to peak of the unit hydrograph was predicted from catchment characteristics using the Flood Studies prediction equation. Some local inconsistencies were found in the relationship between observed and predicted percentage runoff estimated from SOIL type. The latter was thus estimated from the Base Flow Index for catchments with low flow gauging stations and from catchment geology for the remaining areas. These and the time to peak values are listed in Table 9.

The difficulties associated with selecting a single design storm for the whole of the Nene catchment which will result in a 10 year return period flood at all points on the Nene were discussed in Section 7.3. It is suggested that design storms of 70, 60 and 50 mms depth and durations of between 75 and 45 hours (see Table 14) should be used to produce initial inputs to the hydraulic model. These are based on calculations of observed catchment lag to Lilford. If this approach of using a single storm fails to produce a fairly consistent ten year return period flood along the Nene, then separate design storms in the upper and lower parts of the catchment should be used. It is suggested that an acceptable level of consistency will be maintained if return periods (T) along the River Nene are within the range $5 < T < 20$ years.

In order to assess the return period of a design or calibration event it is necessary to compare predicted or observed flows (or levels) with the historically observed flow (or level) frequency curve. This can be done from levels at Lilford and Wollaston and flows at Wansford. It is considered that the volume of flood discharge may be of critical importance and thus frequency curves for durations of 1, 2, 5, 10, 15 and 20 days are shown in Figure 29 for Wansford. It is suggested that these should be used to check the return period of flood inputs routed by the mathematical model to Wansford.

The effect of the Northampton Washland storage scheme on downstream levels and flow frequencies can only be evaluated by using the Nene mathematical model with and without the scheme. By carrying out a number of simulation runs it would be feasible to estimate a revised level frequency relationship at Lilford and Wollaston to take account of the effects of the Washland scheme.

In addition to providing design inputs and a framework for running the Nene mathematical model this study has provided a general hydrological survey of the Nene catchment, which should have wider applications than the immediate scope of this report.

REFERENCES

1. Institute of Hydrology. 'Design flood estimation in catchments subject to urbanisation'. Flood Studies Supplementary Report No. 5, 1979.

2. Natural Environment Research Council, 'Flood Studies Report', Volumes 1-5, 1975.

3. Natural Environment Research Council, 'Low Flow Studies Report', 1980.

APPENDIX B

NENE*GP.CONVPROG

```
C
C         PROGRAM TO CONVOLUTE ANY RAINFALL PROFILE WITH ANY UNIT HYDROGRAPH
C
C         PROGRAM CAN:
C          1. DERIVE DESIGN RAINFALL PROFILE
C          2. DERIVE SYNTHETIC UH
C          3. OR USE OBSERVED RAINFALL AND UNIT HYDROGRAPH DATA
C
C
C         INPUT DATA VIA CHANNEL 5
C         OUTPUTS HYDROGRAPH TO CHANNEL 10
C
      DIMENSION PROF(20),UNIT(120),ERF(100),HRF(100),TITLE(20),
     *CXH(100),CASETI(20),RSFL(300)
      DATA PROF/12.5,24.0,34.5,45.0,53.0,60.0,66.5,72.0,76.0,79.5,82.5,
     *85.0,87.5,90.0,92.0,94.0,96.0,97.5,99.0,100.0/
C
C     AREA=CATCHMENT AREA IN SQ KMS
C     T=DATA INTERVAL IN HOURS
C     D=DESIGN STORM DURATION IN HOURS
C     P=TOTAL RAINFALL IN MMS
C     SPR=STANDARD PERCENTAGE RUNOFF (95.5*SOIL+12.0*URBAN)
C     CWI=DESIGN CATCHMENT WETNESS INDEX
C     SBF=STANDARD BASEFLOW (0.00074*RSMD+0.003)
C     UNIT(J)=ORDINATES OF THE UNIT HYDROGRAPH IN CUMECS PER 100 SQ KMS
C     HRF(I)=ORDINATES OF THE RAINFALL PROFILE IN MMS
C      IPRINC=0 % BASED LOSSES
C      IPRINC=1 LOSS CURVE %RO INCREASES DURING STORM WITH CWI
C
C
      READ (5,100) TITLE
   99 READ (5,101) ICASE,IEV
      IF(ICASE.EQ.0) STOP
      READ (5,100) CASETI
      READ (5,102) AREA,T,D,P,SPR,CWI,SBF
      PR=SPR+0.22*(CWI-125)+0.1*(P-10)
      Q=P*PR/100.0
      ANSF=SBF+0.00033*(CWI-125)
      READ (5,103) NUH,NHRF,IPRINC,TP
      TPNUH=NUH
C
C       NUH = NUMBER OF UH ORDINATES
C       IF NUH>0  READS IN ORDINATES OF UH (CHANNEL 5)
C       IF NUH=0  USES SAME UH AS IN PREVIOUS CASE
C       IF NUH<0  COMPUTES SYNTHETIC UH FROM TP ONLY (QP=220/TP)
C
C       NHRF = NUMBER OF RAINFALL ORDINATES
C       IF NHRF>0 READS IN ORDINATES OF RAINFALL PROFILE(CHANNEL 5)
C       IF NHRF=0 USES SAME PROFILE AS IN PREVIOUS CASE
C       IF NHRF<0 COMPUTES ORDINATES OF 75% WINTER PROFILE
C
      IF(NUH.LE.0) GO TO 3
```

```
      READ (5,104)(UNIT(J),J=1,NUH)
      GO TO 2
    3 IF(NUH.EQ.0) GO TO 4
      QP=220.0/TP
      NUH=INT(2.53*TP/T)+1
      DO 5 J=1,NUH
      IF (J.GT.(TP/T+1))GO TO 6
      UNIT(J)=(QP/TP)*(J-1)*T
      GO TO 7
    6 UNIT(J)=QP*(1-((J-1)*T-TP)/1.53/TP)
    7 IF(UNIT(J).LT.0.0)UNIT(J)=0.0
    5 CONTINUE
      GO TO 2
    4 NUH=NUHB
    2 NUHB=NUH
C
C     NOW HAVE UH ORDINATES IN UNIT(J) WHICHEVER ROUTE USED
C     NOW NEED TO WORK OUT WHICH RAIN PROFILE TO BE USED
C
      IF(NHRF.GE.0)GO TO 19
   10 N=D/T
      PRIB=0.0
   24 DO 11 I=1,N,2
      S=(I*100.0)/N
      S1=S/5
      IS1=S1
      IF (I.EQ.N) GO TO 22
      PRI=PROF(IS1)+(S1-IS1)*(PROF(IS1+1)-PROF(IS1))
      GO TO 23
   22 PRI = PROF(IS1)
   23 PRI=PRI/100.
      IA=N/2+I/2+1
      IB=N/2-I/2+1
      HRF(IA)=(PRI*(1+1/I)-PRIB)/2*P
      HRF(IB)=HRF(IA)
   11 PRIB=PRI
      NHRF=N
      GO TO 12
   19 IF(NHRF.GT.0) GO TO 20
      NHRF=NHRFB
      GO TO 12
   20 READ(5,105)(HRF(I), I=1,NHRF)
   12 RFSUM=0.0
      DO 33 I=1,NHRF
      RFSUM=RFSUM+HRF(I)
   33 CONTINUE
      IF(ABS(RFSUM-P).GT.0.2) WRITE(6,212) P,RFSUM
      IF (IPRINC.NE.1) GO TO 30
      CXH(1)=CWI*HRF(1)
      SCXH=CXH(1)
      AK1=0.5**(T/24)
      AK2=SQRT(AK1)
      IF (CWI.GT.125) GO TO 26
      S=125-CWI
      A=0.0
      GO TO 27
   26 S=0.0
```

```
      A=CWI-125
   27 DO 25 I=2,NHRF
      S=S-HRF(I)
      IF (S.LT.0.0) S=0.0
      A=AK2*HRF(I)+AK1*A
      CWIR=125-S+A
      IF (CWIR.LE.5.0) CWIR=5.0
      CXH(I) =CWIR*HRF(I)
   25 SCXH=SCXH+CXH(I)
      PF=Q/SCXH
   30 DO 13 I=1,NHRF
      IF (IPRINC.EQ.0) GO TO 29
      ERF(I)=PF*CXH(I)
      GO TO 13
   29 ERF(I)=HRF(I)*PR/100.
   13 CONTINUE
      NHRFB=NHRF
C
C     CONVOLUTE ERF(I) WITH UNIT(J) WHERE ERF(I)=HRF(I)*PERCENTAGE RUNOFF
C
      NSFL=NUH+NHRF-1
      DO 8 IJ=1,NSFL
    8 RSFL(IJ)=ANSF*AREA
      DO 9 I=1,NHRF
      DO 9 J=1,NUH
      IJ=I+J-1
    9 RSFL(IJ)=RSFL(IJ)+ERF(I)*UNIT(J)*AREA/1000.0
C
C     FIND PEAK
C
      RSFLM=RSFL(1)
      IJM=1
      DO 15 IJ=2,NSFL
      IF(RSFL(IJ).LE.RSFLM) GO TO 15
      RSFLM=RSFL(IJ)
      IJM=IJ
   15 CONTINUE
C
C           FIND VOLUME
C
      QTP=0
      DO 16 I=1,NSFL
      QTP=QTP+RSFL(I)
   16 CONTINUE
      QTP=QTP*T*3600.0
C
C     FIND CURVATURE OF THE PEAK
C
      IF (IJM.LE.1) GO TO 31
      CURV=(RSFL(IJM+1)+RSFL(IJM-1)-2*RSFLM)/T**2
C
C     OUTPUTS THE RESULTS
C
   31 WRITE(6,200) TITLE
      WRITE(6,201) CASETI
      IF (IPRINC.EQ.1) WRITE (6,209)
      WRITE(6,202) AREA,T,D,P,PR,ANSF,CWI
```

```
      IF(TPNUH.LT.0) WRITE(6,210) TP
      IF(TPNUH.EQ.0) WRITE(6,211)
      IF(NHRF.LT.0) WRITE(6,220)
      WRITE(6,203)
      WRITE (6,204)
      DO 21 I=1,NSFL
      TIME = (I-1)*T
      WRITE (6,205) TIME,RSFL(I)
      IF (I.LE.NHRF) WRITE (6,206) HRF(I),ERF(I)
      IF (I.LE.NUH) WRITE (6,207) UNIT(I)
      IF(I.LE.1.OR.I.GE.NSFL)GO TO 21
      IF(RSFL(I).GT.RSFL(I-1).AND.RSFL(I).GT.RSFL(I+1))WRITE(6,213)
   21 CONTINUE
      WRITE(10,250)ICASE,IEV
  250 FORMAT(2I5,'    071180')
      WRITE(10,251)AREA,T
  251 FORMAT(F8.2,F6.2)
      WRITE(10,252)NHRF
  252 FORMAT(I3)
      WRITE(10,253)(ERF(I),I=1,NHRF)
      WRITE(10,252)NSFL
  253 FORMAT(5F12.4)
      WRITE(10,253)(RSFL(I),I=1,NSFL)
      WRITE (6,208)QTP,CURV
      GO TO 99
  100 FORMAT (20A4)
  101 FORMAT (2I5)
  102 FORMAT (6F8.2,F8.4)
  103 FORMAT (3I5,F5.1)
  104 FORMAT (5F10.4)
  105 FORMAT (8F10.4)
  200 FORMAT (1H1,1X,20A4,///)
  201 FORMAT (2X,20A4,//)
  202 FORMAT (10X,'AREA (SQ.KM.)                     ',F7.2/,
     *10X,'DATA INTERVAL (HR)               ',F7.2/,
     *10X,'DESIGN DURATION (HR)             ',F7.2/,
     *10X,'TOTAL RAIN (MM)                  ',F7.2/,
     *10X,'PERCENTAGE RUNOFF                ',F7.2/,
     *10X,'BASE FLOW (CUMECS PER SQ.KM)',F8.5/
     *10X,'CWI AT START OF STORM       ',F8.2//)
  203 FORMAT (' CONVOLUTION OF UNIT HYDROGRAPH AND NET RAIN PROFILE'/)
  204 FORMAT (2X,'TIME      TOTAL     NET       UNIT          TOTAL',
     */,12X,'RAIN      RAIN      HYDROGRAPH  HYDROGRAPH'/,
     *12X,'MM        MM        ORDINATE      CUMECS   '/)
  205 FORMAT (1X,F6.2,33X,F10.2)
  206 FORMAT (1H+,6X,2F10.2)
  207 FORMAT (1H+,26X,F10.2)
  208 FORMAT (/' TOTAL FLOOD VOLUME (CUBIC METRES) ',F15.3,/,
     1         ' CURVATURE AROUND PEAK                 ',7X,F8.3,//////////)
  209 FORMAT('  PERCENTAGE RUNOFF INCREASING THROUGH STORM WITH CWI')
  210 FORMAT(5X,'TRIANGULAR UNIT HYDROGRAPH COMPUTED FROM TP=',F5.1/)
  211 FORMAT(5X,'UNIT HYDROGRAPH USED FROM PREVIOUS CASE (SEE ABOVE)'/)
  220 FORMAT(5X,'75% WINTER PROFILE USED FOR STORM'/)
  212 FORMAT(1H1,///5X,'*****WARNING***** RAINFALL P NOT EQUAL TO'/,
     *23X,'SUM OF RAINFALL ORDINATES SPECIFIED'/,
     *23X,'P=',F5.1,'SUM OF ORDINATES=',F5.1)
  213 FORMAT(1H+,52X,'-PEAK-')
      END
```

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