\1/017

INSTITUTE of
HYDROLOGY

Barracks Brook

Flood Study

D W Reed and E Field

A Report to Bryant Homes Ltd

November 1986

CONTENTS

Figure captions

Fig.1 Barracks Brook catchment plan

Fig.2 Flood estimates - Barracks Brook (see Section 3.3 for "preferred" estimate)

Fig.3 Flood estimates - Bury Brook

Fig.4 Extract from Anglian WA 1:25000 flood plain map

Fig.5 Representation of storage
- (a) Flood plain cross-section
- (b) Control structure cross-section

Fig.6 Routing of 25-year flood (through combined storage)

Fig.7 Schematic of peak flows for 25-year event
- (a) Combined storage
- (b) Disaggregated storage

Fig.8 Comparison of design hydrographs for 25-year event
- (a) Present condition
- (b) post-development (without balancing)
- (c) Post-development (with balancing)

INTRODUCTION

Outline planning permission has been granted for a substantial residential development on the western fringe of Huntingdon. Bounded by main roads to the south, west and north, and by a main railway to the east, the site is drained by Barracks Brook (see Fig.1).

East of the railway line, Barracks Brook passes through the town centre and discharges into the Great Ouse. The lower reaches of the brook have culverts of limited capacity and present a flooding problem to adjacent property. Thus a condition of the development is that flooding of Barracks Brook should not be further exacerbated.

The Institute of Hydrology was approached by Bryant Homes Ltd to comment on the drainage arrangements proposed by Huntingdon District Council (in conjunction with Anglian Water), with reference to:

(i) the use of historic flood levels to define a natural storage requirement,

(ii) the balancing of runoff from the developed site, and

(iii) the general scheme of storage reservoirs proposed.

The report assesses the characteristics and flood potential of the Barracks Brook catchment, before demonstrating the effectiveness of the proposed retention storages in controlling flows in the design event.

CATCHMENT

The railway embankment immediately east of the site is a key feature. Downstream of the culvert through this embankment, Barracks Brook is an urban watercourse with a known flooding problem. In contrast, upstream of the railway culvert the watercourse has a more rural character and, at present, flooding is not a serious problem.

In the event of an extreme flood arising on Barracks Brook, it is apparent that the limited capacity of the railway culvert (site R on Fig.1) will provide some control to flooding of property downstream. The catchment to

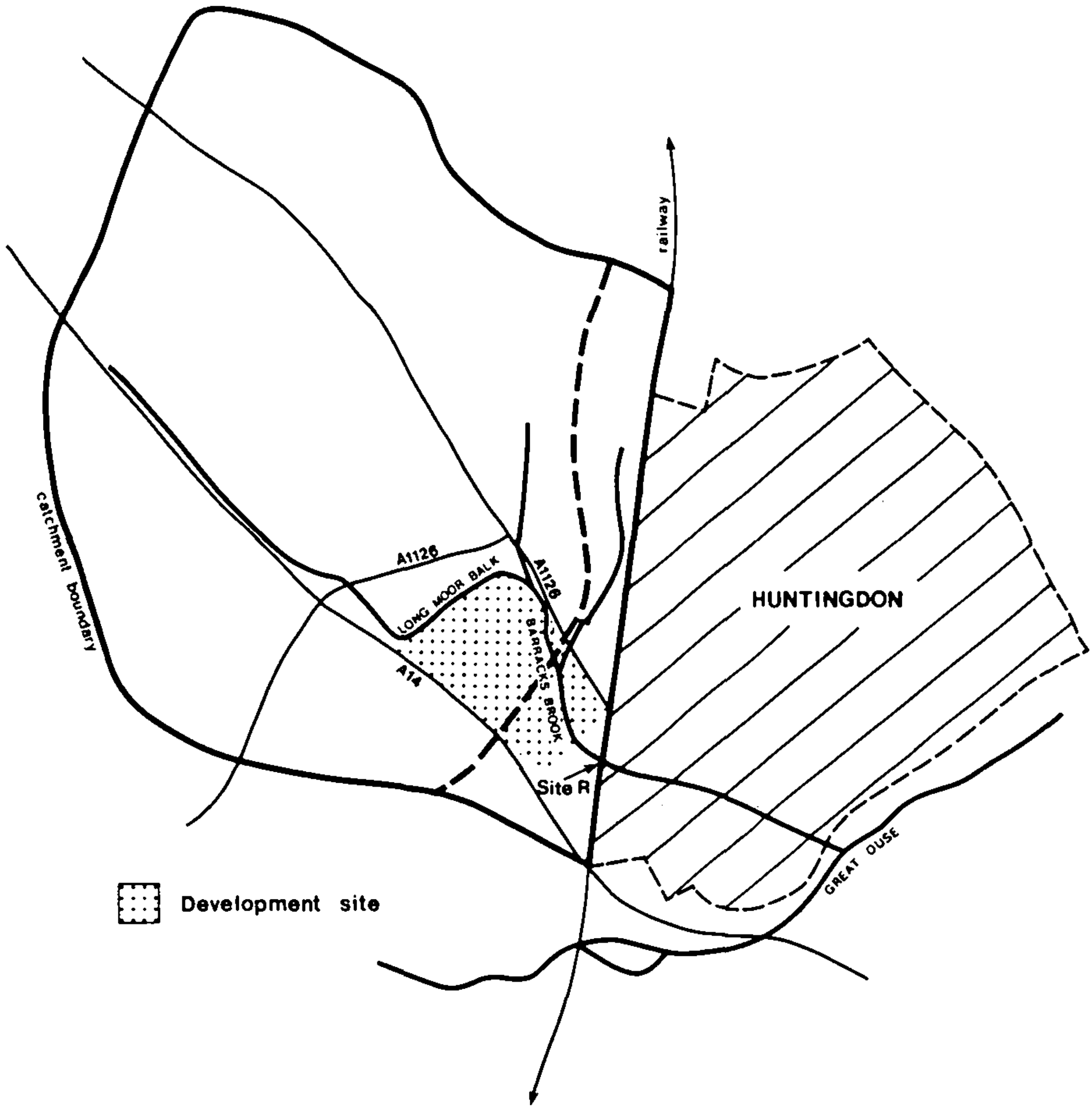


Fig.1 Barracks Brook catchment plan

site R is therefore the logical focus of flood estimates for Barracks Brook.

The catchment to site R is relatively small and flat. The 7.63 km^2 area has an altitude range from about 10 to 47 m AOD, a mainstream length of 3.6 km, and a streamslope of 1.7 m/km (see Table 1). The relevant 1:250,000 soil map shows typical calcareous pelosols of type Evesham 3.

TABLE 1. Catchment characteristics

		Barracks Brook	Bury Brook
AREA	km^2	7.63	65.3
MSL	km	3.59	19.0
S1085	m/km	1.71	1.65
URBAN		0.165	0.020
SOIL		0.404*	0.408‡
LAKE	-	0.0	0.0
STMFRQ	jns/km^2	1.26	1.08
SAAR	mm	560	554
RSMD	mm	20.9	20.2
M5-2D	mm	44.0	44.2
r	-	0.43	0.43

* 92% soil type 3, 8% soil type 4 (but see Section 3.3)

‡ 85% soil type 3, 15% soil type 4 (but see Section 3.2)

Once a largely rural catchment, substantial urbanization has taken place over the last 25 years or so. The village of Great Stukely has expanded considerably, as has Little Stukely - the easternmost part of which now drains to site R. More recently, industrial development has taken place in the east of the catchment and a number of new roads - most notably the A14/A604 bypass have been constructed. The development by Bryant Homes Ltd will further add to the urbanization of the catchment, bringing the urban fraction up to about one sixth.

The effect of urbanization on flood runoff is twofold: greater impermeability increases percentage runoffs, more direct drainage paths accelerate the catchment response. Where urban development is concentrated in the lower reaches of a catchment, it is sometimes the case that a characteristic bimodal response to rainfall occurs: the urban runoff peak departing before the much slower response from the natural catchment arrives. However, the spatial distribution of urbanization in the Barracks Brook catchment is relatively broad and separation of the natural and urban flood peaks cannot be relied upon.

FLOOD ESTIMATES

3.1 Barracks Brook

The partly urbanized nature of the site R catchment makes it appropriate to follow the statistical method of flood estimation given in Flood Studies Supplementary Report No.5 and the rainfall/runoff method given in Flood Studies Supplementary Report No.16.

Calculations for the statistical and rainfall/runoff methods are given in Appendices 1 and 2 respectively. The corresponding flood frequency curves are shown in Fig.2. It is seen that the estimates by the two methods are in moderate agreement.

However flood estimation from "no data" methods is very much less reliable than flood estimation from flow records. Reference was therefore made to a nearby catchment: the Bury Brook at Bury Weir.

3.2 Bury Brook

Although much larger, the Bury Brook catchment is broadly similar to Barracks Brook in terms of soils and topography. The chief dissimilarity is that the Bury Brook catchment is almost entirely rural (see Table 1).

The catchment is one of 175 that have been subjected to standard rainfall/runoff analysis at the Institute (see IH Report No.94 -- copy herewith). Analysis of nine runoff events in the period 1967-1969 noted "standard percentage runoffs" appreciably higher than that inferred from soil maps alone. While the temporal characteristics of the catchment

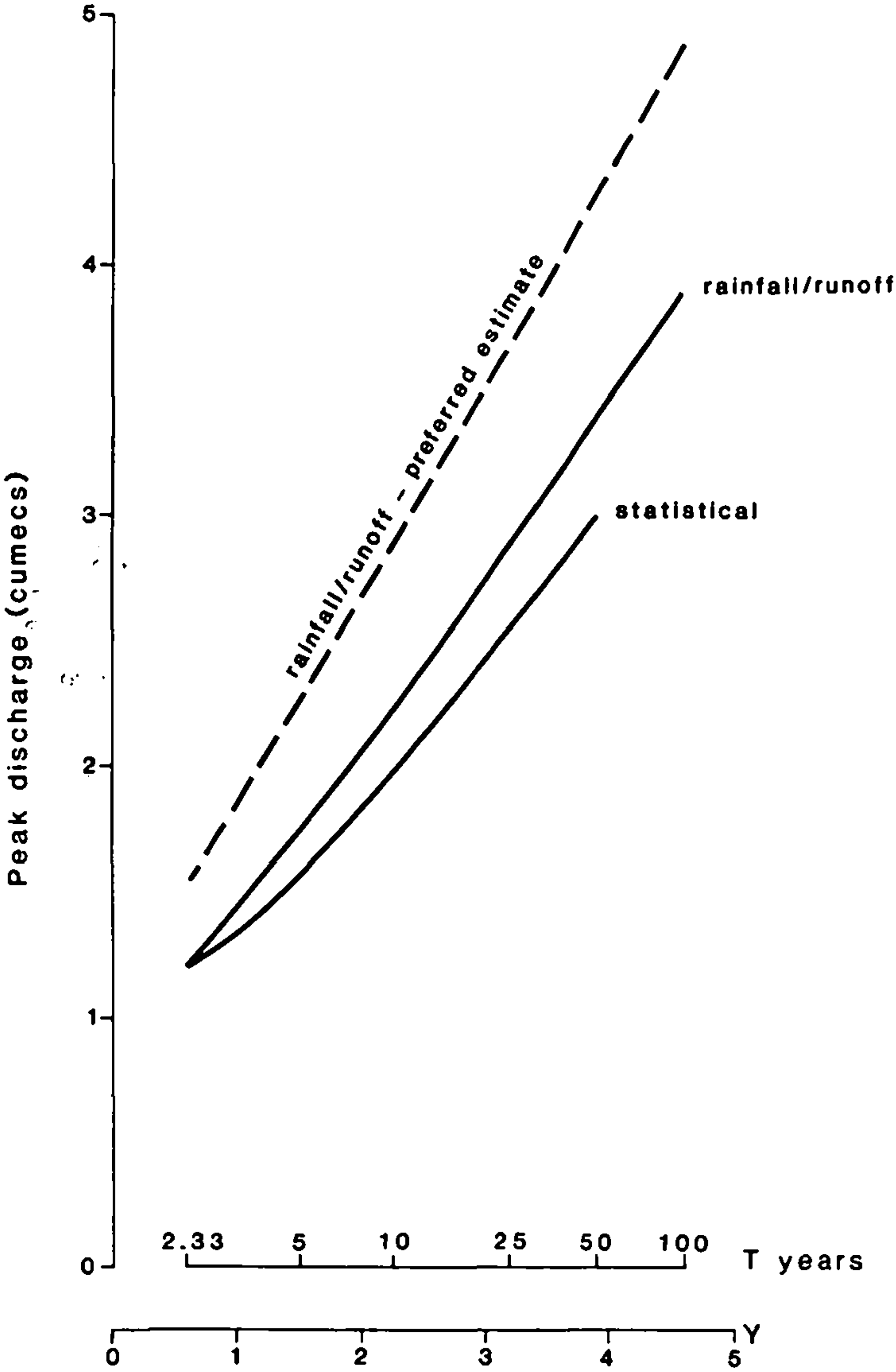


Fig.2 Flood estimates - Barracks Brook (see Section 3.3 for "preferred" estimate)

response agreed well with that estimated by the "no data" method, the net effect of the analysis is to indicate appreciably higher flood frequency (compare "with data" and "no data" rainfall/runoff curves in Fig. 3).

Incorporation of flow data is, of course, possible in the statistical approach also. Analysis of 47 independent peak flows, extracted from 10 years of record (1963/64-1972/73), yielded an estimate for the mean annual flood of 9.7 cumecs. Again, this is significantly higher than by the "no data" method. (Compare "with data" and "no data" statistical curves in Fig.3).

The close agreement, between the rainfall/runoff and statistical methods, adds weight to the conclusion that the flood potential of Bury Brook is appreciably greater than indicated by catchment characteristics. In particular, the soils found in the Bury Brook catchment appear to demonstrate a higher "winter rainfall acceptance potential" index than type 3.

3.3 Adjusted estimates for Barracks Brook

The general similarity of the soils and topography (of the Bury Brook and Barracks Brook catchments) suggests that the "no data" methods may underestimate the flood potential of the Barracks Brook catchment also. However, there are important differences in the catchments in terms of size and degree of urbanization. These lessen the relevance of the information gleaned from the Bury Brook flow data. Nevertheless, some account should be taken of the higher runoff potential of the local soils.

Based largely on experience, a partial adjustment is suggested, re-classifying the soils on the Barracks Brook catchment as 100% soil type 4. This adjustment leads to the "preferred" rainfall/runoff curve shown in Fig.2.

FLOODING HISTORY

While it is known that flooding is experienced fairly frequently in the lower reaches of Barracks Brook - perhaps in a 10 year event - there does not appear to be a regular flooding problem at, or upstream of, site R.

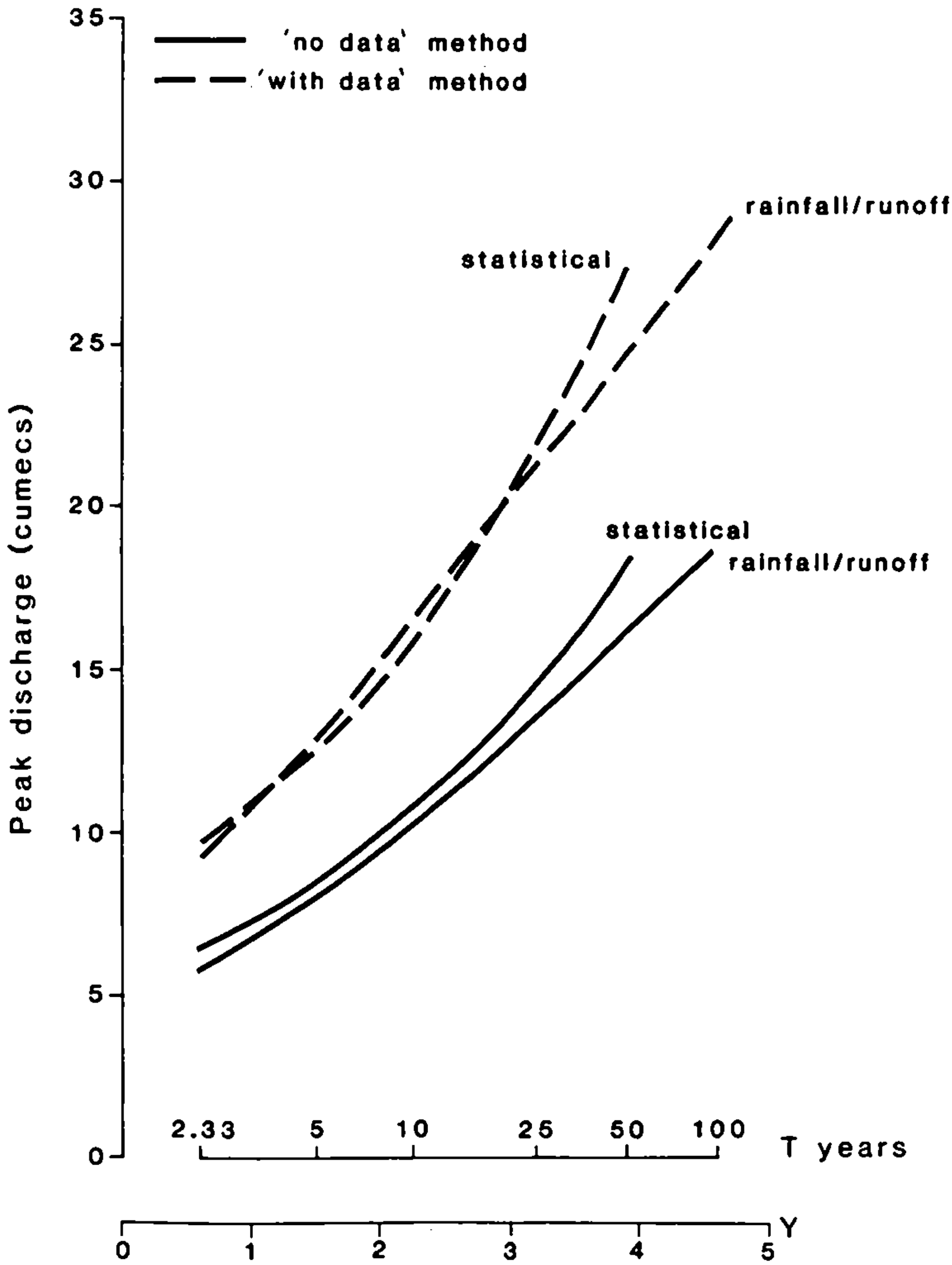


Fig.3 Flood estimates - Bury Brook

Survey data, supplied by Anglian Water for water levels in the widespread severe flood of March 1947, indicate that ponding occurred at site R. However, there appears to be some uncertainty as to the maximum water level reached and considerable uncertainty as to extent of the area inundated.

A further difficulty in assessing the significance of the 1947 event is in judging the extent to which flooding occurred as a result of backing up from the Great Ouse. Water levels on the Great Ouse are well documented and indicate a peak level at the outfall of Barracks Brook of between 9.52 and 9.78m AOD. Given that the site R culvert invert is at 9.00 m AOD (with the soffit at around 10.15 m AOD) it would appear that backing up may well have been a significant factor in the ponding of water at site R.

Assessment of the discharge characteristics of the site R culvert was not included in the study brief. However, cursory inspection indicates an effective aperture of about 2.0 m^2. A back of the envelope calculation for head loss suggests that, with water ponded to the top of the head wall (11.0 m AOD), a mean velocity of about 2 m/s would be achieved, ie. a discharge of 4.0 cumecs. This would to be commensurate both with the flood estimates derived in Section 3.3 and the observation (above) that notable flooding at site R occurs only infrequently.

REQUIREMENT FOR BALANCING

Drawing together the findings of Sections 3 and 4, it is concluded that part of the development site acts as a flood plain, albeit infrequently.

Anglian Water have chosen to define the volumetric extent of the flood plain storage by reference to levels reached in the 1947 event. This is perhaps inappropriate for two reasons.

Firstly, conditions experienced in the March 1947 event were very unusual a widespread severe river flood following rain and heavy snowmelt. Backing up from the Great Ouse undoubtedly contributed to flooding at site R. It is therefore not an obvious design flood to apply to Barracks Brook which is possibly more sensitive to storms of shorter duration.

Secondly, the Barracks Brook catchment is much more urbanized now than it was in 1947. While some of the recent developments have incorporated balancing of storm runoff, this will not be true of earlier developments. However, any underestimation of the flood plain storage required at site R - that use of the 1947 levels might engender - is possibly offset by the relatively generous interpretation of the 1947 water levels made by Bryants. The inundated area assumed in their calculations is very much greater than that shown on Anglian Water's 1:25000 flood plain map (see Fig.4).

That there appears to be no definitive record of the extent of flooding that occurred at site R, means that specification of the 1947 flood volume as a criterion in sizing the balancing requirement is rather vague.

Having said this, the requirement to maintain adequate flood plain storage at site R is indisputable, as is the requirement to balance the increased runoff from the developed site. The proposal to meet these requirements jointly through the use of on-line flood storages appears to be an excellent one, and is discussed next.

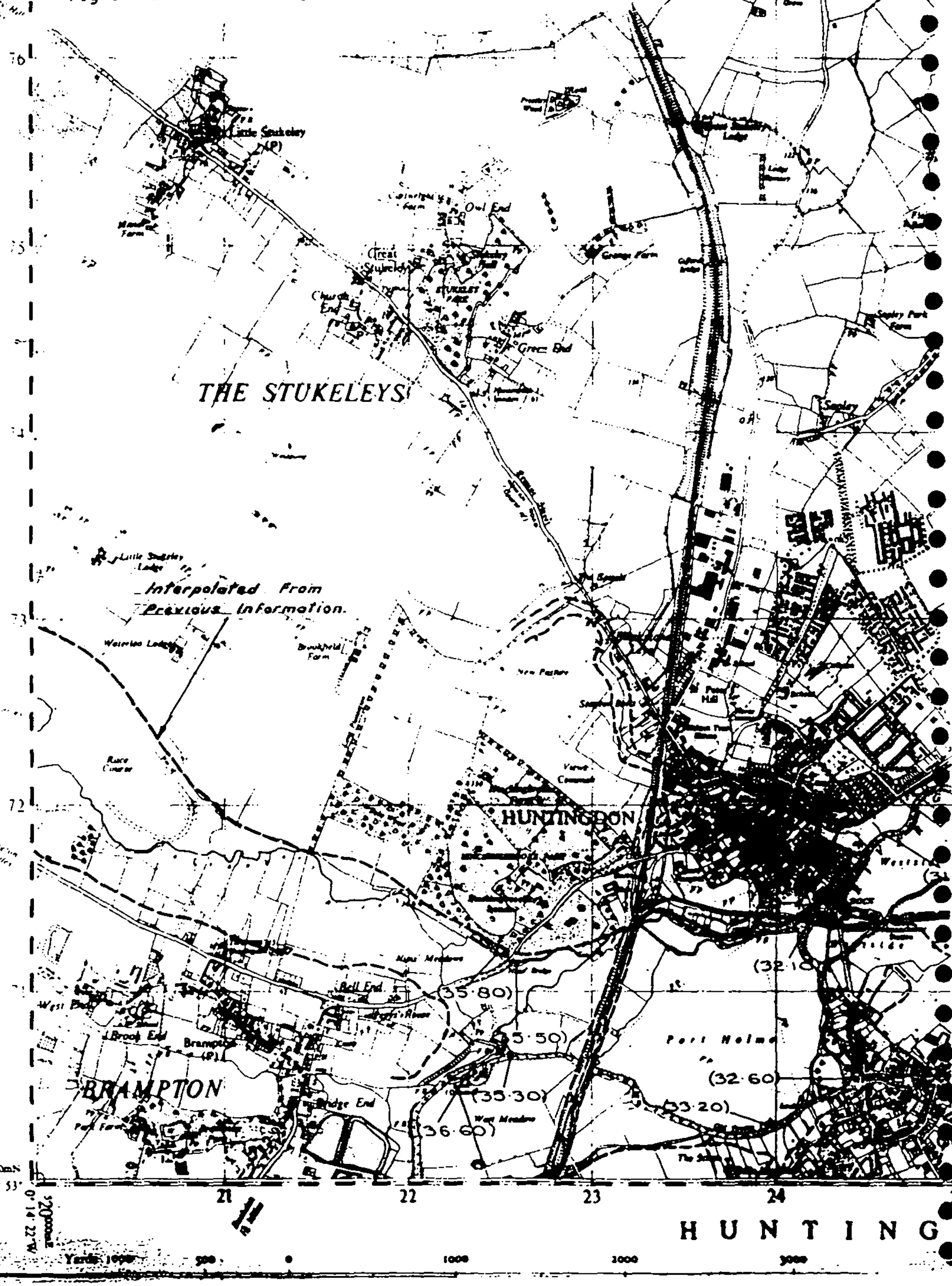
SCHEME OF BALANCING PROPOSED

The scheme of balancing proposed initially (by Huntingdon DC/Anglian WA/Bryants) was for a series of three on-line storages, of which the lowest (and largest) would be formed by a new embankment sited approximately 50 m upstream of site R.

The proposal to use a series of storages appears sensible, given the natural longitudinal slope of the flood plain and the requirement for access roads. However, the construction of the new embankment just upstream of the railway embankment appears to have little merit, and the following drawbacks are seen:

(i) the new embankment would fill-in part of the natural flood plain

Fig.4 Extract from Anglian WA 1:25000 flood plain map



(ii) the sector of natural flood plain between the new embankment and the railway embankment would be blocked off

(iii) the embankment would present an additional obstruction to major floods and, therefore, an additional potential hazard

(iv) as a purpose-built flood retention storage within a capacity far in excess of 25,000 m^3, it would probably be classed as an impounding reservoir and require strict compliance with the many provisions of the Reservoirs Act 1975.

The proposal has the one merit that the new embankment would protect the railway embankment, reducing the frequency with which the latter is required to act as an occasional flood retention structure. However, if there is any concern about the integrity of the railway embankment (to our knowledge, none has been expressed) it would seem preferable to improve protection to the railway embankment directly (rather than via an upstream cofferdam!). It is concluded that construction of the new embankment close to site R should be dispensed with.

As regards the layout and design of the retention storages the following factors are recognised:

(i) there is a need to maintain the storage provided by the existing flood plain (for which an historic flood, such as the March 1947 event, might define a suitable maximum storage)

(ii) there is a need to provide additional storage to balance the increased runoff from the developed area (for which Huntingdon DC/Anglian WA have supplied a criterion) and

(iii) there is a need to "size" the discharge control structures of the upper and middle retention storages so that they come into play at the required frequency.

Factors (i) and (ii) appear to be well understood and values have been agreed independently of this study. However, some flood routing trials assist in factor (iii) and demonstrate the effectiveness of the retention storages in controlling flood flows in Barracks Brook.

FLOOD ROUTING TRIALS

7.1 Introduction

The upper and middle ponds proposed by Bryants have a combined capacity of 20370 m^3 and drain 87% of the catchment to site R. The purpose of the flood routing trials is twofold: firstly, to demonstrate the degree of control exerted on flood flows and, secondly, to assist in correct "sizing" of the control structures.

The design overflow levels of the upper and middle ponds differ slightly (12.0 and 11.8 m AOD respectively) to take benefit from the natural grade of the watercourse. This is, however, a fairly minor difference and, to simplify part of the analysis, the upper and middle storages are considered as a single entity. The analysis determines the size of outlet control required for the middle pond; disaggregation of the results to "size" the control orifice for the upper pond is dealt with in Section 7.5.

7.2 Representation of storage

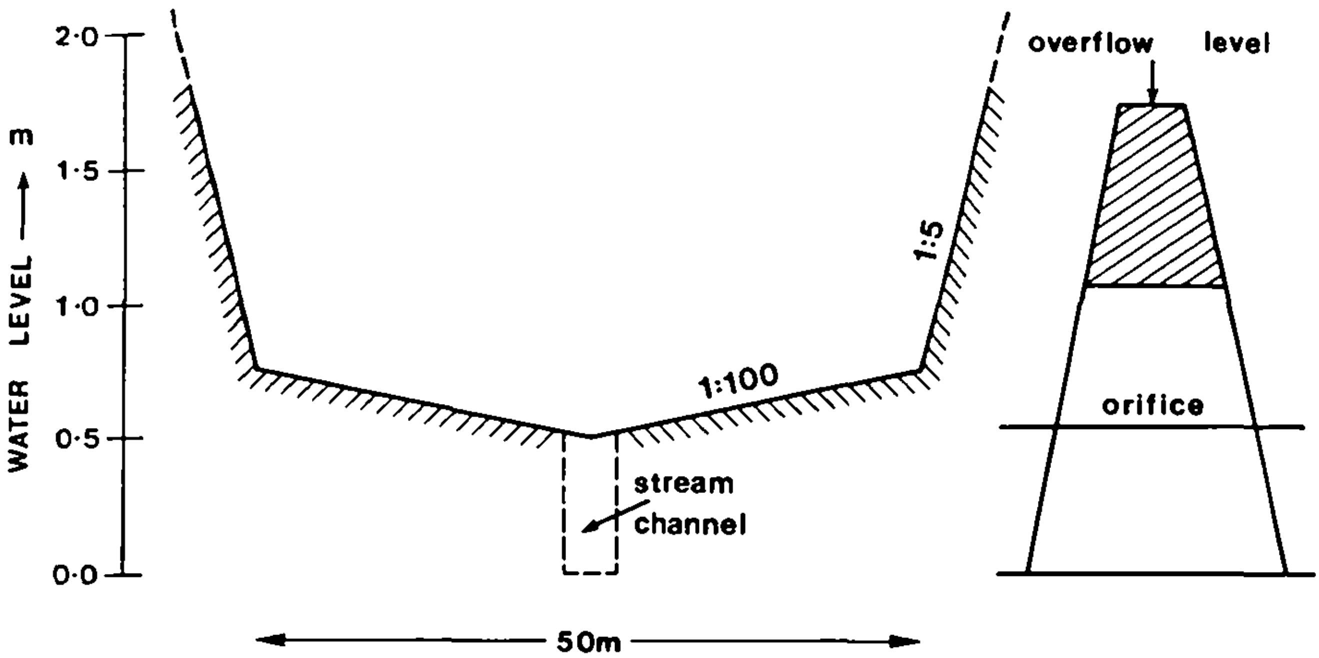
A synthetic representation of the combined storage is possible in the form of a 450 m long, graded flood plain of 1:500 longitudinal slope and a symmetrical 2-stage section (side slopes 1:100 up to a flood plain width of 50 m, with side slopes of 1:5 beyond). This is illustrated in Fig.5. The stream channel itself - assumed to be 0.5 m deep - is excluded from the storage representation. The capacity of the storage to the design overflow level (1.75 m above stream bed) corresponds to the 20370 m^3 of Bryants' design.

Discharge characteristics of the control structure (an orifice or short "throttle pipe") are taken in the form:

(1) $$q = ah^{0.7} \qquad 0 < h < 1.75$$

where q is discharge (cumecs) and *h* is water level (m) relative to the channel invert. (Above h = 1.75 the overflow will operate, and a different rating curve will apply.)

Fig.5 Representation of storage



(a) Flood plain cross-section (b) Control structure cross-section

The design criterion indicated by Huntingdon DC/Anglian WA is that the overflow should operate only for floods in excess of the 25-year event. Thus the parameter "a" is chosen so that the peak water level reached in routing the 25-year event is 1.75 m. The parameter is subsequently interpreted to determine the diameter of orifice required.

7.3 Method

The 25-year flood hydrograph is routed through the storage using a "level pool" method, with storage and discharge characteristics as outlined in Section 7.2. The provision of storage has the effect of attenuating and delaying the flood hydrograph, and a reservoired system therefore tends to be sensitive to longer duration storm events.

Following the procedure recommended in Flood Studies Supplementary Report No. 10, the design storm duration is calculated from:

$$D = \left(1 + \frac{SAAR}{1000}\right)(Tp + MRLAG) \qquad (2)$$

where MRLAG is an areally-weighted mean reservoir lag time. Because only 87% of the inflow to site R passes through the storage, it is appropriate to calculated MRLAG from:

$$MRLAG = 0.87\ RLAG \qquad (3)$$

where RLAG is the ordinary reservoir lag (ie. the time delay between peak inflow and peak outflow).

RLAG is, of course, only known after a trial routing has been carried out. Thus in practice some iteration is required in the calculations.

7.4 Results

The results of the flood routing analysis are summarized in Fig. 6. This shows the inflow and outflow hydrographs for the 25-year event and also the water level variation. It is seems that the peak flow is attenuated by 0.64 cumecs and delayed by 4¼ hours.

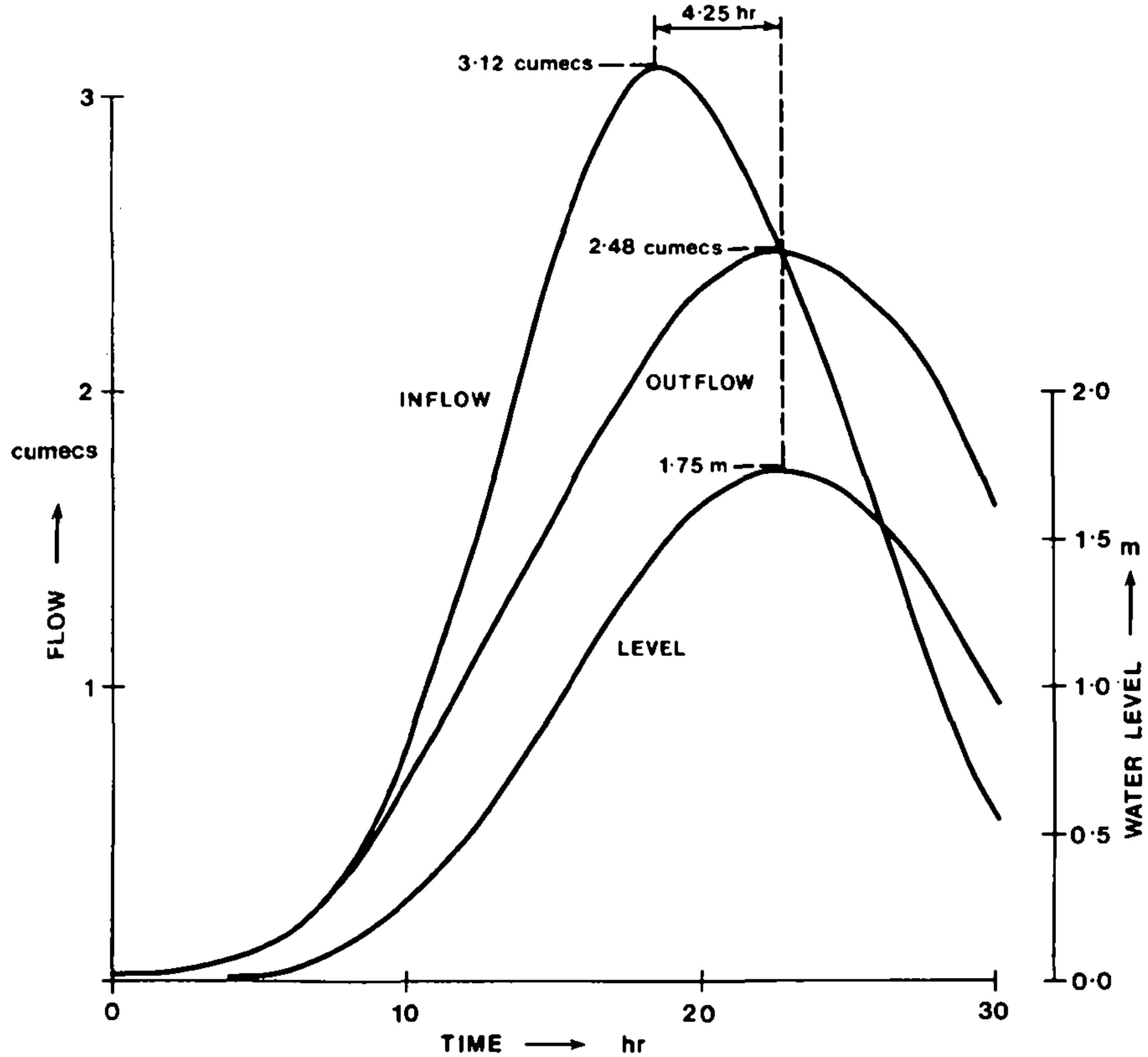


Fig.6 Routing of 25-year flood (through combined storage)

The discharge rating of Eqn. 1 represents the average behaviour over the range of water levels up to 1.75 m. For "sizing" the orifice it is appropriate to apply the formula:

$$(4) \qquad q = c.\frac{\pi d^2}{4}\sqrt{2g(h-\frac{d}{2})}$$

where d is the orifice diameter (m) and c is the discharge coefficient. Taking q = 2.48 cumecs and h = 1.75 m (from Fig.6), and c = 0.6, solution of Eqn. 4 indicates an orifice diameter of 1080 mm.

Addition of the flow contribution, from the 13% of the catchment draining directly to site R, yields a peak flow of 2.87 cumecs. Figure 7a summarizes the make-up of the site R peak flow for the 25-year event.

7.5 Outlet control for the upper pond

A remaining requirement is to "size" the orifice for the upper pond. On the basis that the upper pond provides one third of the attenuation - and drains 90% of the catchment - of the combined storage, the discharge capacity required (QOUT1) can be approximated by:

$$QOUT1 = 0.90\ QIN\ (QOUT/QIN)^{1/3}$$

where QIN and QOUT are the peak inflow and outflow for the combined storage. (See Figs.7a and 7b.) This yields a discharge capacity of 2.60 cumecs, for which an orifice diameter of 1150 mm follows from Eqn.4.

It is preferable that, if anything, the upper pond should spill before the middle pond. Thus a common orifice diameter of 1100 mm would be appropriate for both ponds.

7.6 Discussion

It is of interest to compare the design hydrograph to site R with that

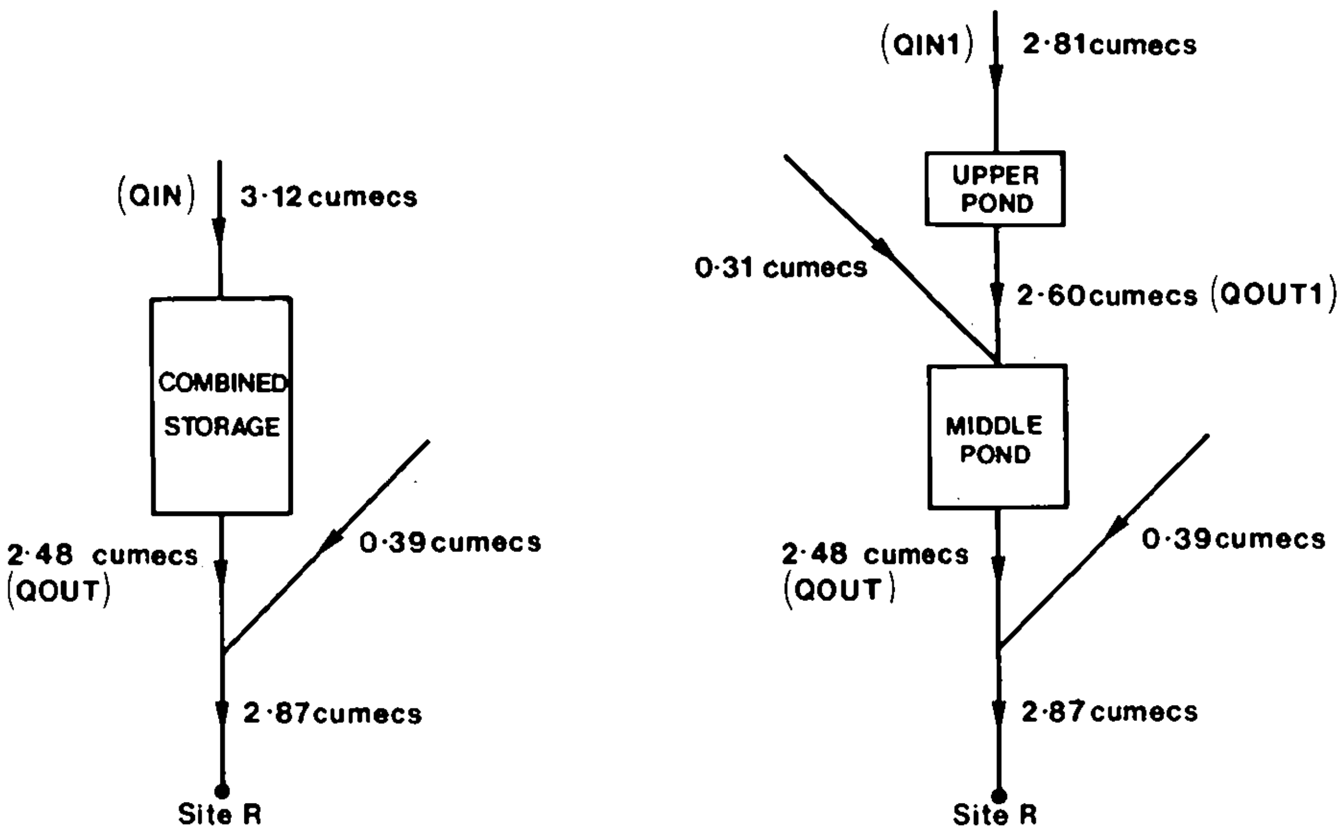


Fig.7 Schematic of peak flows for 25-year event

(a) Combined storage

(b) Disaggregated storage

estimated for the present condition. Appendix 4 shows calculations for the 25-year flood for the present condition. Figure 8 demonstrates that the proposed balancing will significantly control flooding in the 25-year event.

Routing calculations have not been carried out for other return periods. The analysis is intri~~g~~cate and, to an extent, depends on the detailed design of the upper and middle ponds and their discharge arrangements. However, the results obtained indicate that the proposed storage capacity is more than adequate to balance increased runoff from the development.

For floods in excess of the 25-year event, the overflows on the middle and upper ponds will operate and the lower storage area (ie. the land immediately upstream of site R) will come into play. Some inundation of this area may also be experienced in lesser events if significant backing-up from the Great Ouse occurs.

8 FURTHER MATTERS

The study has not explicitly examined the implications of development immediately to the north of Long Moor Balk. This is a smaller development than Bryants' but otherwise of a similar character. Assuming that similar conditions are stipulated by Huntingdon DC/Anglian WA for the provision of storage, this should present no problem to flooding on Barracks Brook. However, it is a general rule that a single large storage is more effective than a series of smaller storages in attenuating floods. Hence it may be appropriate to combine ponds where topography, land ownership, access requirements etc. permit.

Given that, at present, flooding occurs downstream of site R fairly frequently perhaps in a 10-year event the degree of improvement provided by the proposed retention ponds may be entirely appropriate. A further study, examining the flood behaviour of the lower reaches of Barracks Brook, is recommended if any proposal to dispense with one or other balancing area is pursued. This would seem to apply equally to existing and proposed storage areas. However, the effectiveness of two or more large on-line ponds may argue against the retention of small ponds, sited on lesser tributaries of Barracks Brook, if these can be shown to be inconsequential to flood prevention downstream.

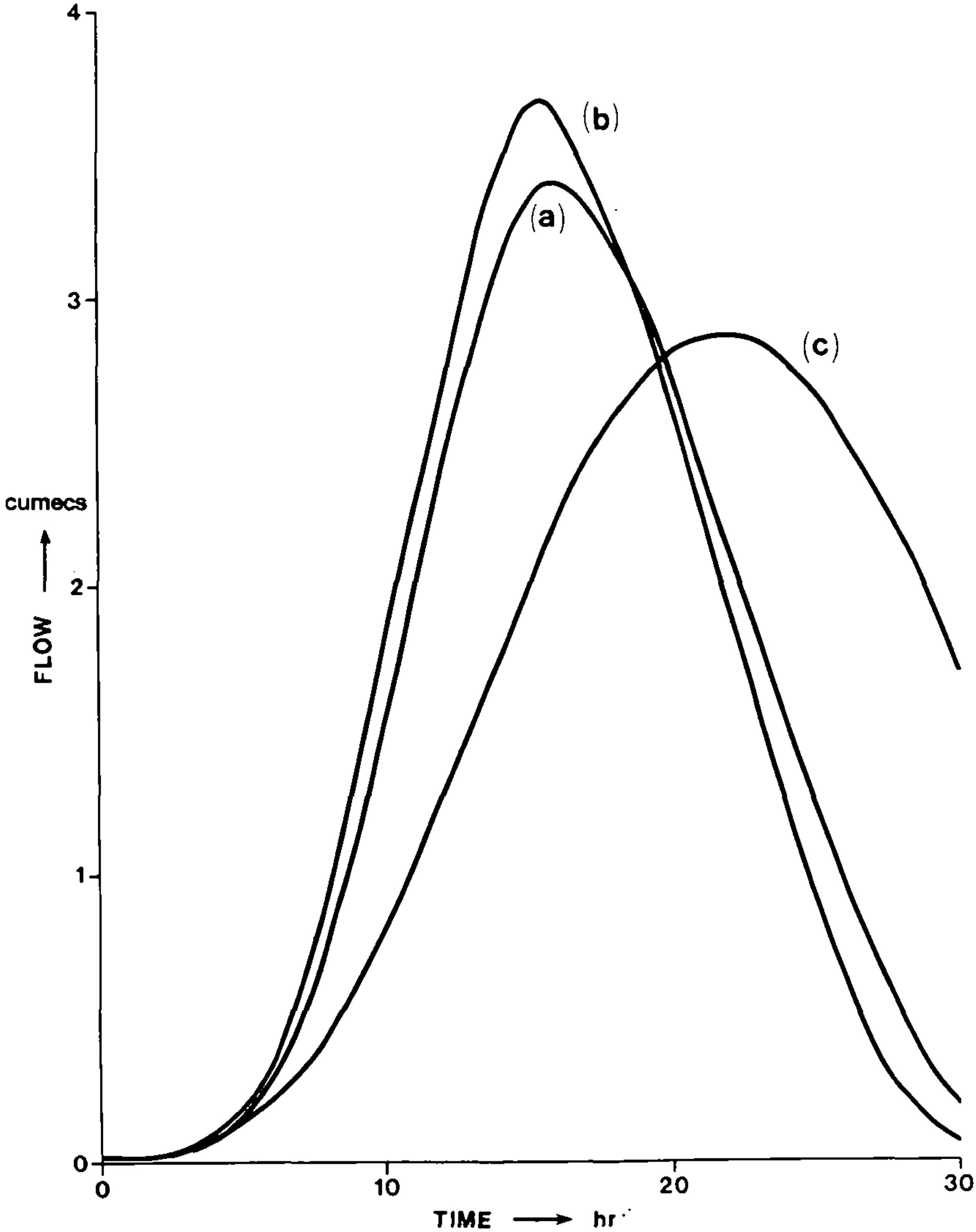


Fig.8 Comparison of design hydrographs for 25-year event

(a) Present condition

(b) post-development (without balancing)

(c) Post-development (with balancing)

CONCLUSIONS AND RECOMMENDATIONS

(i) The general proposals for balancing the effect of the development on flooding downstream are supported, with the caveat that a new embankment immediately upstream of the railway culvert should not be constructed.

(ii) Correct sizing of the control structures on the upper and middle ponds is important. Orifice diameters of 1100 mm are recommended. Substantial overflows are recommended to deal with floods greater than the 25-year event.

(iii) The proposed upper and middle ponds will reduce the frequency of flooding at, and downstream of, the railway culvert. (See Fig.8.)

(iv) Further investigation of flood behaviour downstream of the railway culvert is recommended before any proposal to dispense with existing balancing provision is pursued.

DR/vw

27.11.86

APPENDICES

APPENDIX 1.

Statistical 'no data' method - Barrocks Brook.

$$\bar{Q} = \text{Const AREA}^{0.94}\,\text{STMFRQ}^{0.27}\,\text{S1085}^{0.16}\,\text{SOIL}^{1.23}\,\text{RSMD}^{1.03}\,(1+\text{LAKE})^{-0.85}$$
$$= 0.0153(7.63)^{0.94}(1.26)^{0.27}(1.71)^{0.16}(0.404)^{1.23}(20.88)^{1.03}(1.0)^{-0.85}$$
$$= 0.9\ m^3/s$$

Effect of urbanisation on mean annual flood (FSSR 5)

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

$\bar{Q}_r \quad \bar{Q} = 0.9\ m^3/s$

$$PR_r = 102.4\ \text{SOIL} + 0.28(\text{CWI} - 125)$$
$$= 102.4(0.404) + 0.28(78.3 - 125)$$
$$= 28.29$$

$$\frac{\bar{Q}_u}{\bar{Q}_r} = (1+ \cdot 165)^{1.5}\left\{1 + 0.3(0.165)\left(\frac{70}{28.29} - 1\right)\right\}$$
$$\frac{\bar{Q}_u}{\bar{Q}_r} = 1.35$$
$$\bar{Q}_u = 1.22\ m^3/s$$

R.I. (years)	Growth factor	$\bar{Q}_u$ (m^3/s)
2.33	1	1.2
25	2.11	2.5
50	2.49	3.0

Growth factors are dependent on URBAN and region (Region 5).

APPENDIX 2

****

UK DESIGN FLOOD ESTIMATION PACKAGE   Institute of Hydrolog

****

Run reference - hunt

Catchment characteristics

Area	7.63	sq km	Soil 1	0
Length	3.59	km	Soil 2	0
Slope	1.71	m/km	Soil 3	0.92
SAAR	560	mm	Soil 4	8e-02
M5-2D	44	mm	Soil 5	0
Jenkinson's r	0.43			
Urban	0.165			
Smdbar	14.3	mm	RSMD 20.88	mm

Unit hydrograph option

Unit hydrograph time to peak   7.96 Data interval
Design storm duration 13 hours

Return Period for design flood   25.0 years
requires rainfall event depth of 42.5 years

M5-2day = 44 mm
M5-13.0hour = 34.9 mm
M 42.5-13.0hour = 56.1 mm (point)
M 42.5-13.0hour = 54.4 mm (area)

M5-13.0hour/M5-2day = 0.79
MT/M5 = 1.61
ARF = 0.97

Rainfall profile option 4

Design storm depth 54.4 mm

Design CWI 78.3

Percentage runoff 31.1 %   (PR option 1

Response hydrograph peak   2.85 cumecs
Baseflow   0.01 cumecs   (Baseflow option 1)

Design hydrograph peak   2.86 cumecs

****

APPENDIX 3

```
***************************************************************************
UK DESIGN FLOOD ESTIMATION PACKAGE                       Institute of Hydrology
***************************************************************************

                                              Run reference - hunt

---------------------------------------------------------------------------
Catchment characteristics
---------------------------------------------------------------------------

Area             7.63          sq km      Soil  1           0
Length           3.59          km         Soil  2           0
Slope            1.71          m/km       Soil  3           0
SAAR             560           mm         Soil  4           1
M5-2D            44            mm         Soil  5           0
Jenkinson's r    0.43
Urban            0.165
Smdbar           14.3          mm         RSMD   20.88      mm

---------------------------------------------------------------------------

Unit hydrograph option

Unit hydrograph time to peak   7.96 Data interval
Design storm duration 13 hours

Return Period for design flood     25.0 years
requires rainfall event depth of  42.5 years

                                          M5-2day = 44 mm
M5-13.0hour/M5-2day = 0.79                M5-13.0hour = 34.9 mm
MT/M5 =   1.61                            M 42.5-13.0hour = 56.1 mm (point)
ARF = 0.97                                M 42.5-13.0hour = 54.4 mm (area)

                                          Rainfall profile option   4

Design storm depth  54.4 mm

Design CWI  78.3

Percentage runoff 39.8 %          ( PR option  1

Response hydrograph peak          3.65 cumecs
Baseflow                          0.01 cumecs        ( Baseflow option  1

Design hydrograph peak            3.66 cumecs

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

APPENDIX 4

```
**************************************************************************
UK DESIGN FLOOD ESTIMATION PACKAGE                    Institute of Hydrology
**************************************************************************

                                            Run reference - hunt

--------------------------------------------------------------------------
Catchment characteristics
--------------------------------------------------------------------------

Area             7.63        sq km      Soil 1           0
Length           3.59        km         Soil 2           0
Slope            1.71        m/km       Soil 3           0
SAAR             560         mm         Soil 4           1
M5-2D            44          mm         Soil 5           0
Jenkinson's r    0.43
Urban            0.12
Smdbar           14.3        mm         RSMD   20.88         mm

--------------------------------------------------------------------------

Unit hydrograph option

Unit hydrograph time to peak   8.63 Data interval
Design storm duration 13 hours

Return Period for design flood     25.0 years
requires rainfall event depth of  42.5 years

                                        M5-2day = 44 mm
M5-13.0hour/M5-2day = 0.79              M5-13.0hour = 34.9 mm
MT/M5 =  1.61                           M 42.5-13.0hour = 56.1 mm (point)
ARF = 0.97                              M 42.5-13.0hour = 54.4 mm (area)

                                        Rainfall profile option  4

Design storm depth  54.4 mm

Design CWI  78.3

Percentage runoff 39.4 %          ( PR option  1

Response hydrograph peak          3.39 cumecs
Baseflow                          0.01 cumecs         ( Baseflow option  1 )

Design hydrograph peak            3.40 cumecs

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