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Flood estimation in mixed urban/rural catchments

Final Report on the Bracknell catchment case study

MAFF Project FD0413

J. C. Packman and E. J. Hewitt

Institute of Hydrology
Crowmarsh Gifford
Wallingford
Oxfordshire
OX10 8BB
UK

Tel: 01491 838800
Fax: 01491 692424
Telex: 444293 ENVRE G

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Summary

The potential impacts of urbanisation on flood volumes, response times and peak discharges are well recognised, and most urban areas now include costly flow reduction measures in the form of 'balancing' reservoirs, tanks and ponds. However, detailed observations of urban catchment response or the effectiveness of flood storage provisions have rarely been made, particularly in catchments that include a mixture of land uses or a number of storage ponds distributed over the area. Assessing the flood alleviation needs for such catchments requires a thorough understanding and proven modelling capability covering:

- the interaction of flood response patterns from different land-uses and development types,
- the effects of sewer hydraulics, flow controls and localised flooding, and
- the effect of areal, temporal and seasonal variability in rainfall and soil conditions.

These requirements have remained largely unfulfilled due to the poor extent and quality of observed rainfall-runoff data in mixed catchments.

The objectives of this project, as discussed in Chapter 1, were comprehensively to monitor and model the flood response of the Cut at Binfield, a mixed urban/rural catchment comprising forest, pasture and urban land-use, and encompassing the new town of Bracknell and a network of flood storage tanks and ponds. The model studies would account for hydraulic aspects of flood response, but would also determine the impact of rainfall and soil conditions in order to advance the selection of appropriate hydrological conditions for design use. A broad discussion of the problems of flood modelling in mixed catchments is given in Chapter 2 of this report, and Chapter 3 describes the establishment of the four year flow monitoring programme in the Bracknell catchment. Considerable effort has been spent trying to develop consistent data sets, but although response time information is good, some of the volumetric data remains more indicative of catchment behaviour than absolute. Chapter 4 gives a brief overview of SCHEME, a semi-distributed model developed at the Institute of Hydrology specifically for mixed catchments.

The application of the model is presented in Chapter 5, based on 31 flood events extracted from the observed record, with 16 events used for *fitting* and the other 15 events used for *testing*. The goodness-of-fit achieved with SCHEME has been compared with the lumped Flood Studies Report (FSR) model and at each stage of the analysis SCHEME has consistently given better results:

- with optimum parameters, derived for each model and *fit* event <u>individually</u>, SCHEME gave an average correlation coefficient, R^2, of 0.95 compared with 0.91 for the FSR model:
- with overall 'best-fit' parameters, based on assessing all the *fit* events <u>together</u>, SCHEME gave an average R^2 of 0.94 compared with 0.90;
- with predicted parameters, based on relationships found between the 'best fit' parameters, Soil Moisture Deficit and rainfall duration, SCHEME gave an average R^2 of 0.94 compared with 0.89;
- with predicted parameters, using the same relationships, but applied to the *test* events, SCHEME gave an average R^2 of 0.94 compared with 0.84.

Similar results were also obtained based on other measures of goodness-of fit.

The study has shown that, by treating urban and rural areas separately, SCHEME can match both the rapid urban rise time and the slower rural response; the FSR model could only approximate these features by adopting an excessively skewed unit hydrograph, with time to peak just 8% of the full time base rather than the normal 40%. In terms of identifying urban impacts, both models found that runoff response time varied with Soil Moisture Deficit, with a slower overall response observed in wetter conditions when pervious area response formed a more significant component of total flow.

As well as giving a better representation of overall catchment response, SCHEME also provided a good fit to observed response within the catchment. It could thus be used with confidence to determine the effect of the balancing ponds. This showed that the 15 main balancing ponds had reduced peak flows at Binfield by an average of 22% over all the *fit* events, but by 33% for the largest *fit* event. This compared with an apparent increase in mean annual flood at Binfield between the periods 1957-1973 and 1974-1990 of 56%.

It is concluded that SCHEME provides a working method for assessing mixed urban/rural catchments and estimating the impacts of hydrological variability and hydraulic interventions. A number of recommendations are however made for improving SCHEME and including it within a T-year flood estimation procedure.

Two long Appendices are included with this report. Appendix A describes the theoretical basis of SCHEME, and Appendix B describes the Bracknell catchment and hydrological data archive.

In addition to MAFF funding, this study had the support and co-operation of The EA Thames region, Thames Water Utilities, Bracknell Forest Borough Council, and ADS Environmental Services (flow survey contractors).

1. Objectives and justification

The original objectives of this project were:

(1) to develop methodologies for flood estimation in mixed urban/rural catchments, particularly for catchment planning applications,
(2) to verify such methodologies through a comprehensive case study of the Bracknell catchment, including the effects of local drainage throttles, flood control/attenuation measures, areal rainfall patterns, and mixed seasonal response characteristics, and
(3) to generalise design conditions through studying the relative frequency of individual flood events at different locations in the catchment.

These objectives were aimed at real needs in drainage design. Uncontrolled urbanisation increases flood runoff and raises mean annual flood by typically 200-500%. The increase depends on a range of factors, including local catchment features, drainage details, and local seasonal characteristics. The dependence of urban impact on catchment and drainage features has been assessed previously (e.g. Packman, 1980; Hall, 1984; Marshall and Bayliss, 1994), but on a lumped catchment basis rather than by considering the impact of localised changes within the catchment. The effect of different seasonal sensitivities in flood generation (extensive paved areas generating maximum runoff from intense summer rainstorms, pervious areas yielding maximum floods from 'saturated' soils in winter) has never been properly assessed. The need remains for flood estimation models that are based both on a sound understanding of hydraulic and hydrologic response, and on a proper assessment of the range of conditions throughout the annual cycle that can cause flooding.

Urban flood protection and the alleviation of downstream impacts currently involves a combination of channel improvements and flood storage. New strategies of controlling urban runoff 'at source' (e.g. SEPA/EA, 1997) have not yet been widely applied in the UK and are not within the scope of this report. However, at the larger end of source control are the flood storage ponds (sometimes called by some permutation-combination of: detention, retention, balancing, retarding, storm, pond, basin, reservoir, or [underground/sewer] tank). Storage ponds are now included in new urban developments almost as a matter of course, and are thus distributed throughout most urban areas. Existing design guidance, however, is based on single ponds (e.g. Hall *et al.*, 1993), and makes minimal reference to ponds in combination. Moreover, despite their considerable cost, the effectiveness of storage ponds, singly or combined, in reducing flooding has never to the authors' knowledge been properly appraised through field study as opposed to model study (which reflects the model structure rather than that of the real catchment-storage system). Indeed, few ponds have even been surveyed or their control structures rated 'as built'.

As discussed in Chapter 2, assessing the flood response of mixed catchments by combining current models of the hydraulics and hydrology of urban and rural catchments can lead to inconsistencies. The models have developed separately, based on different objectives and different (limited) data sets. The fundamental objectives of this project were to identify and help resolve such inconsistencies, using data collected via a detailed case study of the Bracknell Catchment.

The Bracknell catchment was chosen because (a) the urban area was clearly defined and almost wholly within a single catchment, (b) a core hydrological network already existed, and (c) the catchment included a broad range of land uses and various flood control structures.

Calibration and extension of the existing gauge network has however lead to difficulties in developing accurate data series (see Chapter 3 and Appendix B). Particular problems concerned the rating of level recorders, and the monitoring of long term response for storage ponds and small single land-use subcatchments. For these reasons, objectives (2) and (3) were relaxed in October 1997, as follows:

(2a) to verify the methodologies through a case study of the 'Bracknell' catchment, including broad comparisons of flow response at specific flood storage ponds and from selected urban and rural subcatchments, and

(3a) to provide (i) a broad indication of generalised design conditions using available data, and (ii) a discussion of the range of problems to be considered when designing/evaluating urban drainage systems.

Despite errors and uncertainties, the data collected still give a unique picture of runoff processes through a mixed urban/rural catchment, and provide valuable information on the timing and general shape of flood response. They currently provide firm evidence of the capability of models to predict observed response patterns within the catchment. Ways in which the data may be further improved are discussed in the conclusions and recommendations to this report.

2. Background

2.1 GENERAL

Urban land use and traditional drainage systems reduce infiltration and increase the volume and speed of runoff. Flood rise times are reduced, typically by about 75%, and flood peaks increased, typically by 200% or more at the mean annual flood level - see Flood Studies Supplementary Report No.5 (FSSR5, Institute of Hydrology, 1979) and Marshall and Bayliss (1994). In mixed catchments, flood response from an urban area covering 25% or more of the catchment will normally exceed that from the rural area, and dominate the response of the catchment as a whole. The scale of these impacts, however, depends on many factors, including:

- the underlying natural response of the catchment;
- the distribution and type of urban surfaces;
- the form of the urban drainage system and its state of maintenance, and
- the seasonal variation of the local climate.

In particular:

- Catchments characterised by low runoff and sluggish natural response will yield proportionately greater impacts of urbanisation than catchments already giving high and rapid runoff.
- Urbanisation distributed over less responsive parts of a catchment will yield proportionately greater impacts on overall flood runoff (though impacts may be moderated through the use of semi-permeable pavement and local soakaway drainage).
- Urbanisation of headwater areas will tend to accelerate local runoff yielding coincidence and reinforcement of slower response from downstream rural areas, whereas downstream development may allow the urban response to pass before the upstream rural response arrives.
- The urban drainage system (a 'minor system' for 'de-watering' the urban area) may redefine catchment divides and flow paths, and under flood conditions may 'surcharge' causing local choking, upstream flooding, and downstream pressure surges.
- Specific flow structures, such as flap valves, vortex orifices, pumping stations, overflows, and on/off-line tanks or ponds may divert or attenuate flows, possibly under real-time control, altering the phase and scale of response from different parts of the catchment. Channel and structure maintenance (or lack thereof) may also have a significant impact on drainage system operation.
- As urban surfaces yield runoff even when soils are dry, urban runoff response is relatively insensitive to seasonal changes in soil moisture conditions, and high intensity summer thunderstorms pose the greatest risk of flooding. In rural areas, the buffering effect of dry summer soils means summer runoff is generally low and flooding is a predominantly winter phenomenon.

Flood estimation in mixed urban/rural catchments must therefore account for background hydrology, engineering hydraulics, and also the seasonal disparity in runoff generation and storm conditions.

As the flood response of a mixed urban/rural catchment is likely to be dominated by urban runoff, its accurate determination is crucial for assessing flood alleviation needs in both local and receiving watercourses. Urban runoff would arguably be most accurately estimated by an urban drainage (sewer) model capable of modelling surcharging and flow controls (e.g. the WASSP, WALLRUS and HydroWorks family of models - see DoE/NWC, 1981; Wallingford Software, 1989, 1994). However, such models have been developed largely independently of rural models and are based on different knowledge bases, modelling standards, and design criteria. Flows are usually determined at many locations within small (less than 10 km^2) but complex drainage systems that may include sewage and surface runoff. Great spatial detail (subareas of 1 ha or less) and short timesteps (1-15 secs) are required to model rapidly varying flow conditions. With advances in computer hardware, software and GIS database technology, the models can be applied to larger catchments that may include mixed urban/rural areas - though rural areas are usually treated simplistically. Urban drainage models are intended for designing drainage to restrict localised flooding to (typically) about a five year return period, and for assessing the impact of relatively frequent storm overflows and discharges to receiving waters (typically ten or more overflows per year).

By contrast, rural models (e.g. the Flood Studies Report; NERC, 1975) usually consider fewer design points and longer timesteps (15-60 minutes), and they cannot represent the urban response in detail. They are usually concerned with rarer conditions (of fifty years or more return period), when urban drainage systems operate beyond their design limits (and beyond the verified capability of the drainage models).

Simply combining urban and rural models, therefore, either directly or through their outputs, may result in over-complexity without improving veracity. Moreover, as most urban and rural models contain implicit seasonal bias in their runoff generation (with urban models developed and calibrated on summer data and rural models mainly on winter data), combining models may also overlook the seasonal effects or may inadvertently combine worst case scenarios that would never coincide. Examples include:

- the neglect of winter runoff conditions in urban runoff models, leading to undersizing of flood storage tanks in mixed catchments, and
- higher percentage runoff (PR) predicted by the Flood Studies Report (FSR) equation for rural conditions than is sometimes predicted by the HydroWorks equation for urbanised conditions, implying that urbanisation has <u>reduced</u> percentage runoff.

This latter anomaly arises because the HydroWorks equation was derived wholly from summer runoff events in small sewer catchments, and for predictive use adopts summer antecedent conditions, whereas the FSR equation was derived mainly for large rural catchments, from runoff events of which three quarters occurred in winter, and adopts design antecedent conditions that are essentially winter. A more plausible interpretation is that rural PR for large river catchments in <u>winter</u> can be higher than urban PR for small sewer catchments in <u>summer</u>. The problem has been partly resolved in HydroWorks by considering winter and summer antecedent conditions in the PR model, though the wetter winter conditions will always give a worst case unless some adjustment is made for seasonal rainfall differences. FSR Vol. 2 (p30) indicates that short duration summer rainfall is typically 1.6 to 2.3 times greater than winter rainfall for the typical design durations of less than 6 hours; no account of such seasonal rainfall differences is currently made.

It is sometimes claimed that the need to consider appropriate design rainfall and antecedent conditions could be avoided by adopting continuous simulation methods. Such methods could also help clarify design criteria for flood storage ponds for which mixed criteria are often specified at present (for example: balance the 10-year volume flood to the once a year peak flood prior to urbanisation). However, besides requiring appropriate long-term hydrometric data sequences, continuous simulation also brings a considerable computational and data management requirement, and is better suited to verifying final designs than to developing initial solutions. A need for simple design storm methods persists. Moreover, although continuous simulation may address seasonality in rainfall and antecedent conditions, it will not resolve model inconsistencies arising from the separate development and calibration of urban and rural components. Fitting a model to continuous periods of generally low-flow conditions will not necessarily improve its fit to the highest peaks. In any model study results may reflect the model structure more than the real catchment behaviour, but with continuous simulation there may be a greater risk of unquestioning belief in the model.

Improved flood estimation in mixed urban/rural catchments depends on combining urban and rural models that cover the full range of flow conditions, incorporating in a user-friendly way the ability to model development patterns and hydraulic structures, (including behaviour under high flood conditions), and also including a proper consideration of runoff generation and storm characteristics. The development of such integrated urban/rural modelling must rely on observations, both of runoff hydraulics (through pipes, storages and flood ponds) and of rainfall and soil hydrology. There is however, a general lack of recorded data on runoff patterns in mixed or fully urbanised catchments - over storm, seasonal and annual time scales.

For urban runoff processes, existing public data in the UK derives mainly from studies of 2-3 years duration in 17 small (less than 2 km^2) fully sewered catchments (Makin and Kidd, 1979). None of these catchments included controls such as storage ponds, tanks or overflows (the overall performance of which is still poorly understood). In most urban drainage studies, model performance is verified against a short term (6 week) flow survey, comprising typically 30-50 flow monitors and 3-5 raingauges distributed over the sewer system. These surveys are primarily intended as a coarse check on catchment description rather than as an investigation of hydrological phenomena. Flow survey data have not been fed back into model development. Apart from the private nature of the data, measurement difficulties and limited quality control generally result in low accuracy (usually taken as ±25%), and the short duration of survey gives little information on rarer events.

For larger (less than 100 km^2) mixed catchments typical of catchment planning studies, our knowledge of how the processes combine is scarce. Only 5 catchments in the UK Representative Basin Network are both less than 100 km^2 and more than 25% urbanised, and none of these have separately monitored urban and rural subcatchments. The development of flood models for mixed land-use catchments requires data from a network of flow gauges to verify that urban and rural response patterns are being combined correctly over the seasonal cycle (or beyond), and that the impacts of flood control measures on local and wider catchment response are being properly assessed.

The lack of observed data for proving mixed catchment models may in part reflect a perception of urban flooding as a costly, but localised and largely non life-threatening phenomenon. It may also reflect the difficulty of monitoring urban catchments, where:

- swollen flows swamp existing channels and measuring structures,
- flash flow responses require a short data interval (less than the 15-minutes typically adopted for rural gauging),
- hydraulic and sediment conditions yield flow monitoring difficulties,
- access to suitable gauging sites is restricted or dangerous, and
- population density increases the risk of vandalism.

Moreover it may reflect the organisational structure of the UK water industry: a strategic network of long-term high quality gauging stations is maintained for large rural rivers by the Environment Agency, but urban flow monitoring is usually private to the Water Utilities, consisting of the short-term surveys (discussed above) that are vital for operational purposes, but have not contributed to improvements in modelling science.

Continuing organisational changes may also have obscured problems. Besides the change in river gauging authorities (from River Authorities to Regional Water Authorities, to the National Rivers Authority and now the Environment Agency), there have been similar changes in drainage authorities. In Bracknell, for example, the main balancing ponds were designed, built and maintained under the direction of the Bracknell Development Corporation, transferred to Bracknell Forest Borough Council acting as agents for Thames Water Utilities, but are now managed directly by Thames Water (in each change no local staff were transferred, and Bracknell Forest meanwhile has become a unitary authority). Subsequent ponds and tanks have been designed by consultants and developers; some have been 'adopted' by Thames Water while others remain privately owned (and maintained). Moreover, there are several significant lakes in the grounds of private estates, where attitudes to maintenance have not been consistent. Raised backwater due to higher levels in one such lake has been blamed for some incidences of flooding. No definitive list has been maintained of the location of flood storage facilities in the Cut catchment, let alone what their dimensions are.

To summarise, there is a gap both in gauged experience and in modelling capability for medium-sized, mixed urban/rural catchments. Effective flood protection requires that urban and rural subcatchment responses are considered separately and in combination throughout the seasons, and that the effects of flood control measures are considered in their local and downstream impacts. Of particular concern are the interaction of different land-uses, development types and response patterns; the effects of sewer hydraulics, controls and localised flooding; and the effect of areal, temporal and seasonal variability in rainfall and soil conditions. Resolution of these issues depends on observation and understanding, providing the justification for the Bracknell catchment case study.

2.2 THE BRACKNELL STUDY

Bracknell new town covers approximately 30% of the 50 km^2 catchment of the Cut at Binfield (see Fig. 3.1), with extensive development continuing around the Bull Brook, a tributary to the north of the town. Drainage is by separate sewers for foul and surface runoff, with 15 significant flood storage ponds in the surface system. The river drainage system also includes three private ornamental lakes and a large pond. Outside the urban area, land use in the catchment comprises mainly forest and pasture, with some villages and estates.

The aim of the field study was to monitor rainfall-runoff response in a variety of different types of catchment as follows:

- in the upstream Cut (essentially rural),
- at the two main and one minor urban 'outfalls' into the Cut,
- at inlets and outlets to two balancing ponds (one on-line, one off-line) within the urban area, and
- in two small rural catchments (one forest, one pasture).

By monitoring 'good' urban and rural sites over several years, and through improved data processing, continuous and contemporaneous flow records would be obtained throughout a mixed catchment. These records were to be used in paired/nested catchment studies to determine urban impacts on flood magnitude and frequency on an individual storm and seasonal basis.

A full description of the catchment and monitoring programme follows in Chapter 3, while further information on data processing is given in Appendix B.

Support for the Bracknell study (in addition to the direct MAFF sponsorship) has come from Bracknell Forest Borough Council (BFBC), Thames Water Utilities (TWU), Thames Region of the Environment Agency (EA, formerly the National Rivers Authority), and the flow survey contractors ADS (now franchised to IHS, a former subsidiary of HR Wallingford, but now independent). The study was not linked to any operational projects, though the EA had carried out modelling studies of the Cut using a now obsolete version of the FRQSIM model, and had been concerned with increased channel erosion downstream of Bracknell. BFBC/TWU had had a number of problems with surface flooding and the maintenance of storm tanks; they had modelled surface water in the 2 km^2 Great Hollands subcatchment, and were planning a full Drainage Area Survey of the foul system. Recent new development east of the Bull Brook had included a new balancing pond (Jiggs Lane), but a *no balancing* strategy was adopted for the west (to avoid multiple ponds or inverted siphons under the Brook to Jiggs Lane pond). Analysis of urban flood impacts and the performance of balancing ponds was seen as a great benefit to the understanding of recognised problem areas. Ponds were costly to build and their overall efficiency was unknown. There was also concern with erosion in the Cut downstream of Bracknell. All the agencies agreed that Bracknell was an ideal location for this study.

MAFF funding provided eight flow loggers, two level recorders, and a contract for the operation of 3 ADS flow loggers. The ADS loggers had been recommended by HR Wallingford as the best available, and were operated by ADS (on 'loss leader' terms) in the main 'triple bore outfall' from Bracknell to the Cut. It should be noted that the widespread use of multi-bore pipes in Bracknell increased the number of monitors needed for the study and affected the choice of storage ponds to monitor.

Support from EA Thames Region included the provision of five flow loggers, three depth recorders, four raingauges, and the management of much of the field data collection programme. They also provided data from the telemetering raingauge at Bracknell STW (Sewage Treatment Works), from the flow gauge on the Cut at Binfield, and from five 'temporary level recorders' installed at locations on the Cut since the mid 1980s. In addition they provided survey data for the older storage ponds (as collected for the FRQSIM study).

TWU, through their (then) sewerage agents BFBC, provided information on sewerage networks within the catchment. This included a copy of their STC25 digital database (DoE/NWC, 1980) giving pipe locations and dimensions, allowing the identification of likely monitor locations and the definition of drainage boundaries. They also provided a 'sewer gang' and safety equipment to accompany all field trips involving entry into the drainage system, and authorised the installation of 'intrinsically safe' monitors (that couldn't produce a spark in a confined atmosphere) and secure fixings (probes on bands screwed to pipes, loggers installed below manhole covers or in adjacent pits). Finally, they provided additional local knowledge and information on many balancing ponds and sewer tanks.

In addition to the field study discussed above, the aim of the Bracknell study was to develop guidance on modelling urban impacts in mixed catchments, and to determine the design conditions that should be considered when estimating flood magnitude and frequency throughout the catchment. Such guidance could be based on one of a number of models built on a 'linked subcatchment' structure, notably the American models TR20 (Soil Conservation Service, 1982) and HEC-1 (Hydrologic Engineering Centre, 1990), the Australian Model RORB (Laurenson and Mein, 1988), and the UK models FLOUT/RBM (HR Wallingford, 1989), FRQSIM (EA Thames Region, 1994), and SCHEME (Appendix A). The overseas models are fully commercial packages, but have rarely been used in the UK. FLOUT/RBM is also a commercial package, currently being replaced by a newer HR Wallingford model (ISIS), but which had some limited use in the UK, while FRQSIM and SCHEME are essentially proprietary models for in-house use. Any of the models could probably have been used (with varying degrees of difficulty and modelling success).

The SCHEME model has been adopted as the basis for this study because:

- it was developed at the Institute of Hydrology specifically for modelling mixed urban/rural catchments,
- it is designed for ease of fitting and analysing hydrological data, and
- research findings on subcatchment response and design conditions could probably be transferred to the other models (the UK ones in particular).

SCHEME (a Sub-Catchment Hydrological Event Model for Engineers) has represented a strategy as much as a specific model form, and has developed since the late 1970s through application on a range of research and consultancy projects. The model is a natural extension of the FSR methods, combining the recommended catchment, river channel and reservoir routing models as components within a 'linked subcatchment' framework. The model includes the distribution of raingauge data to subcatchments (based on grid references), and allows the fitting of observed flow hydrographs by adjusting 'factors' on the standard FSR model parameter equations, an approach analogous to the 'local data' recommendations of FSSR13 (Institute of Hydrology, 1983). As it stood at the start of this project, apart from a few general upgrades, the only significant extension required to embark on objectives (1) and (2) as given in Chapter 1 was the addition of a procedure for modelling off-line storage ponds. The model does not include an explicit pipe-routing facility, but a version of the original WASSP Sewered Subarea Model (Price *et al.*, 1980) could easily be incorporated. A summary of the modelling basis of SCHEME is given in Chapter 4, with fuller details in Appendix A.

In this study, detailed modelling of sewer processes has not been adopted. If however sewer flow processes (including surcharge/pressure flow) had proved of greater significance, an alternative modelling procedure based more explicitly on sewer processes could have been adopted. The standard UK sewer models (HydroWorks and the now obsolete WALLRUS) are only available as (costly) commercial packages that cannot be extended by 'third parties'. They do not include rural catchment modelling, but do allow direct hydrograph inputs at sewer inlets, and give output hydrographs at sewer outfalls. A framework could be devised to generate rainfall and hydrograph inputs to such a model, run the model, and then collect the output hydrographs for input to a wider catchment model (but fitting the model to observed data would involve much repetition). Alternatively, a version of the earlier WASSP model, which has been updated at the Institute of Hydrology to include a rural catchment model and some of the WALLRUS modelling features, could form a more integrated basis for modelling. It may be noted that at the start of this project in 1992 many engineers still used WASSP, though TWU had moved to WALLRUS for its ability to model the effect of sedimentation on pipe flow. HydroWorks was released in late 1994, and by the end of the project, use of WASSP had virtually ceased.

3. The research catchment - overview

3.1 PHYSICAL FEATURES

The catchment and main drainage system of the Cut to Pitts Weir, Binfield are shown on Fig. 3.1, together with land use taken from OS 1:10000 maps (see Appendix B). The catchment area, given as 50.2 km^2 in the National Water Archive, comprises flat to gently-rolling pasture and woodland, and includes most of the New Town of Bracknell, covering an area of about 15 km^2. From its source in the east of the catchment (near Ascot Race Course) the Cut flows north and then westward along a mainly rural course, falling 34.5 m to Pitts Weir over a length of 11.95 km. Its two main tributaries are steeper, the partially urbanised Bull Brook running northward through the centre of the catchment has a fall to Pitts Weir of 55.5 m over 11.67 km, and the more westerly and more urbanised Downmill Stream has a fall of 53.5 m over 6.87 km. Soils in the catchment are reflected in the (rural) land use, with the *Holidays Hill* and *Southampton* series (sands and gravels over clay, well drained but some seasonal waterlogging) in the wood/heath areas, and the *Wickham* series (loam over clay, slowly permeable with some seasonal waterlogging) in the pasture area. These soils are classified as type 3 and type 4 respectively in the Flood Studies Report (FSR). Mean Annual Rainfall, according to the National Water Archive is 687 mm, and the 5-year return period one hour rainfall depth from the FSR maps is 20.3 mm.

Development of Bracknell New Town started with the central and northern areas between 1950 and 1960. This was followed by the south-west areas from 1965 to 1980, the eastern fringe from 1980 to 1990, and the northern fringe from 1990 to date. Like most New Towns, Bracknell is drained by separate surface runoff and sanitary sewers: only the surface drainage has been considered in this study. Figure 3.2 shows the storm sewer network taken from Bracknell Forest Borough Council's (BFBC) sewer database (see Appendix B) overlain on the IH digitised river network (from the OS 1:50000 maps), indicating the increased drainage density within the urban area. The eastern side of Bracknell drains to the largely open channel Bull Brook, the western side to the now culverted Downmill Stream, and a small northern part drains directly to the Cut between the two tributaries. In the far south-west of the catchment, a 1.9 km^2 area of housing and woodland that originally drained westward to the Emm Brook has been diverted by the urban drainage system into the Cut catchment (some recent small developments do still drain to the Emm Brook, via storage ponds). The catchment also contains Ascot Sewage Treatment Works (STW), discharging to a tributary of the Bull Brook at point L on Fig. 3.2.

Within the surface drainage system there is a large number of storage ponds, lakes and tanks. Table 3.1 lists 30 'ponds' for which some information was available from either EA Thames Region or BFBC. Other farm ponds and lakes are shown on OS maps, and there are thought to be further private sewer tanks on some of the newer commercial developments. The first 20 ponds from Table 3.1 are shown on Fig. 3.2, but only the first 18 have been modelled in this study. Information on storage and outflow relationships was not available for the other ponds, and it was considered beyond the scope of this study to collect it, particularly since their effect on flow at the selected monitor locations (given below) would be small. More specifically:

- No.19 (Englemere Pond), despite being the largest pond, drains a comparatively small, flat rural/wooded area of 94 ha. It has a free outfall not designed to choke the flow, and outflow must also pass through the Ascot Place lake downstream.

Land use upstream of the Cut at Binfield

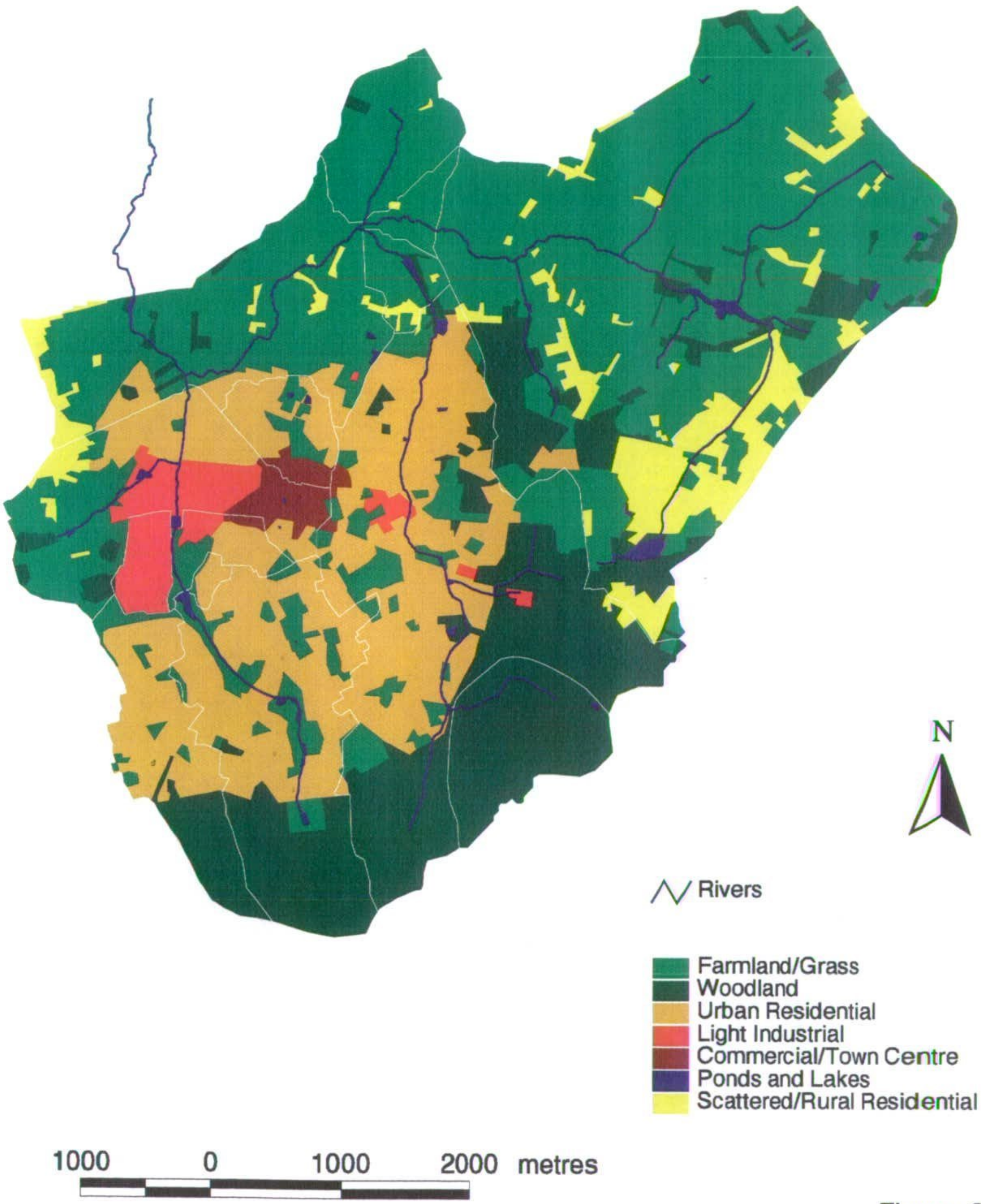


Figure 3.1

River network, storm sewers and storage ponds

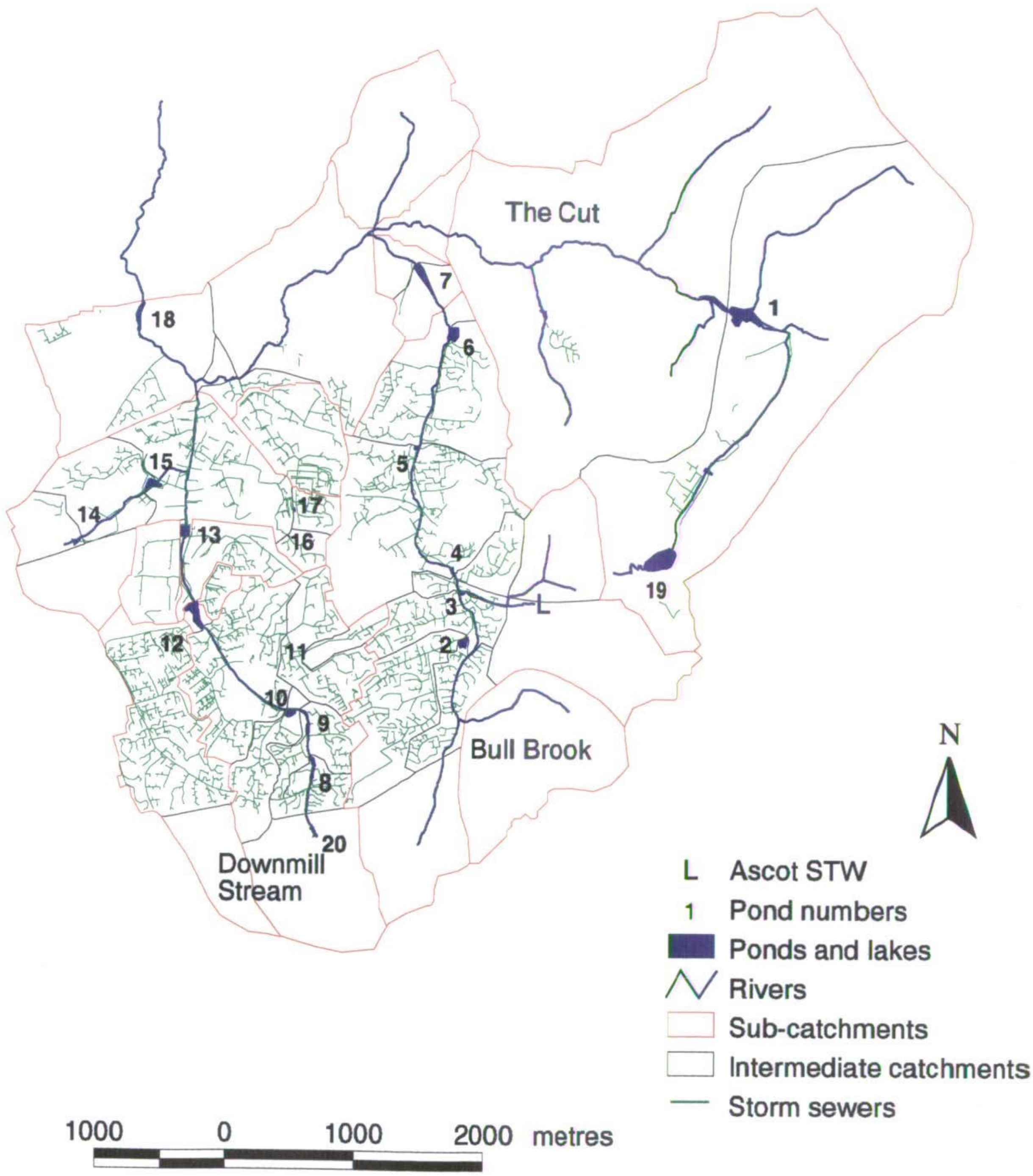


Figure 3.2

- No.20 (Gormoor Pond) similarly drains a rural/forest area of 126 ha, with a free outfall, and has the South Hill Park ponds downstream.
- No.21 (Carnation Nursery) is a small pond draining a small area of about 14 ha, and will thus have a small effect on the overall catchment response.
- The remaining ponds (nos. 22-30) are small, normally draining a single enterprise, and in some cases it is doubtful they have even been built.

Note also:
- Three of the ponds (11, 16 and 17) that were modelled are also small, but had storage and outflow data readily available, and also their outflows contributed to flows monitored relatively close-by.
- Where depth:area data were not available, particularly for the three ornamental lakes (no.1: Ascot Place, no.7: Warfield House, and no.18: Binfield Lake), pond area was taken as constant and determined from the digital 1:10000 maps (see Appendix B). At Ascot Place, a weir of 4 m length was used to represent the 'artificial rock bar' that formed the real outlet control.
- Although a similar 'best guess' approach could have been used to assess the likely impact of the Englemere and Gormoor Ponds, it was felt that their effects were likely to be small.

Table 3.1 Storage ponds and lakes.

No	Pond name	Pond type	Map Ref.	Area (ha)	No	Pond name	Pond type	Map Ref.	Area (ha)
1	Ascot Place	ornamental	915712	5.50	16	St. John's Ambulance	on-line, dry	868689	0.06
2	Savernake Pond	on-line, wet	887678	1.02	17	Multi-storey Car Park	on-line, dry	869692	0.08
3	The Warren	off-line, dry	886683	0.19	18	Binfield Lake	ornamental	853712	2.10
4	Martins Heron	on-line, wet	885686	0.77	19	Englemere Pond	ornamental	907688	5.90
5	Bay Road	off-line, dry	882698	0.25	20	Gormoor Pond	ornamental	872659	0.49
6	Jiggs Lane	on-line, wet	884709	1.00	21	Carnation Nursery	on-line, wet	895710	0.07
7	Warfield House	ornamental	882706	2.10	22	Crouch Lane	dry	922729	-
8	South Hill Park 1	on-line, wet	871667	0.69	23	Fernbank Road	wet	904691	-
9	South Hill Park 2	on-line, wet	870671	0.85	24	Doncastle Road	dry	858687	-
10	South Hill Park 3	off-line, dry	868671	0.57	25	Doncastle Road	dry	858687	-
11	Sports Centre	on-line, wet	870677	0.06	26	Staplehurst	dry	852666	-
12	Mill Pond	on-line, wet	859682	2.63	27	Coral Reef 1	-	878663	-
13	Oldbury	off-line, dry	859690	1.05	28	Coral Reef 1	-	879664	-
14	Amen Corner	on-line, wet	848688	0.51	29	Bloomfield Drive	-	876702	-
15	Waterside Park	on-line, wet	855695	1.34	30	Sainsburys	-	878667	-

All the storage ponds designed specifically for flood control date from the later period of urban development (after about 1965) and appear to have been designed in isolation from each other. Most of the storages are on-line structures, the performance of which could be assessed by conventional (level pool) reservoir routing models. Four of the modelled ponds are off-line with storm flow diverted over side weirs. The performance of such ponds is poorly understood - particularly when they fill and the weir starts to drown. It may be noted that the term 'off-line' is used throughout this report to mean flow is diverted into the storage from a low-flow bypass channel; the term is used by some authorities to describe ponds that are on the line of a tributary or collector channel, but off the line of the main catchment drain (e.g. Jiggs Lane, no.6 on Fig. 3.2, which receives all the runoff from the east of the Bull Brook below Bay Road, no.5, and then discharges direct to the Brook). Fuller details of the 18 ponds modelled in this study are given in Appendix B. The total flood storage volume available in these storage ponds is approximately 210,000 m^3.

3.2 HYDROLOGICAL MONITORING STRATEGY

The hydrological monitoring network established for this study (see Fig. 3.3) has been developed from a number of previous and continuing monitoring networks in the Cut catchment. Flow data have been collected at the Binfield weir (point A on Fig. 3.3) since 1957, covering most of the period of expansion. Data since 1986 are held by the EA in computer form at 15-minute intervals. From 1975-1980, flow data were collected on chart recorders at flumes installed in the outfalls from three urban subcatchments draining to the Mill Pond (B). These data were used in developing WASSP, the Wallingford Storm Sewer Package (DoE/NWC,1981), but data reliability was low and the gauges were abandoned. A fourth flume was also installed in the outflow culvert, but the data collected were never processed as the flume was permanently 'drowned' by the effect of a downstream constriction. In 1986, EA Thames Region installed five 'temporary' river level gauges to support a catchment study. The original chart recorders were converted to shaft encoders or pressure transducers in 1988, recording data at 15-minute intervals. Three of these gauges were still operational during this project: Wane Bridge (C) on the Cut; and Warfield House (D) and the Weir site (E) on the Bull Brook. If these gauges could be rated and the data converted to discharge, a reasonable length of record would be available for frequency analysis.

The strategy of this project in terms of data collection for a comprehensive study of the response of a mixed catchment was thus (*n.b.* letters in parentheses refer to Fig. 3.3):

- to access the EA flow data for Binfield (A), and level data for both Wane Bridge (C - on the mainly rural part of the Cut), and Warfield House (D - on the Bull Brook just downstream of the urban area).
- to monitor flow depth and velocity at (or near) Wane Bridge and Warfield House in order to develop rating equations.
- to monitor flow at Jocks Lane (F - the main surface water outfall of the culverted Downmill Stream, draining the western side of Bracknell)
- to monitor inflows and outflows at one on-line balancing pond. The Mill Pond (B), was the 'obvious' choice, re-instrumenting the flumes on the Easthampstead, Great Hollands and Wildridings inlets (draining residential areas of 5.2, 2.3 and 0.1 km^2 respectively), and monitoring the outflow at some suitable point.
- to monitor inflows and outflows at one off-line balancing pond. Oldbury Pond (G) was chosen, comprising a side-weir channel with two upstream inlets (Industrial and Waitrose) and a downstream throttle pipe (Bypass). The Waitrose inlet drains an industrial area of 0.5 km^2, and the (so called) Industrial inlet drains a mixed residential and industrial area of 0.6 km^2. With the pond outflow controlled by twin 200 mm pipes, affording poor access and suggesting much lower drainage rates than Bypass flows, and with limited instrumentation available, it was decided to concentrate on the weir channel, and neither water level in the pond, nor drainage from the pond was monitored.
- to monitor runoff from a town centre/commercial area of high impermeability. The Benbricke Green (H) outlet to the Cut was chosen, giving additional information on

Hydrological monitoring network

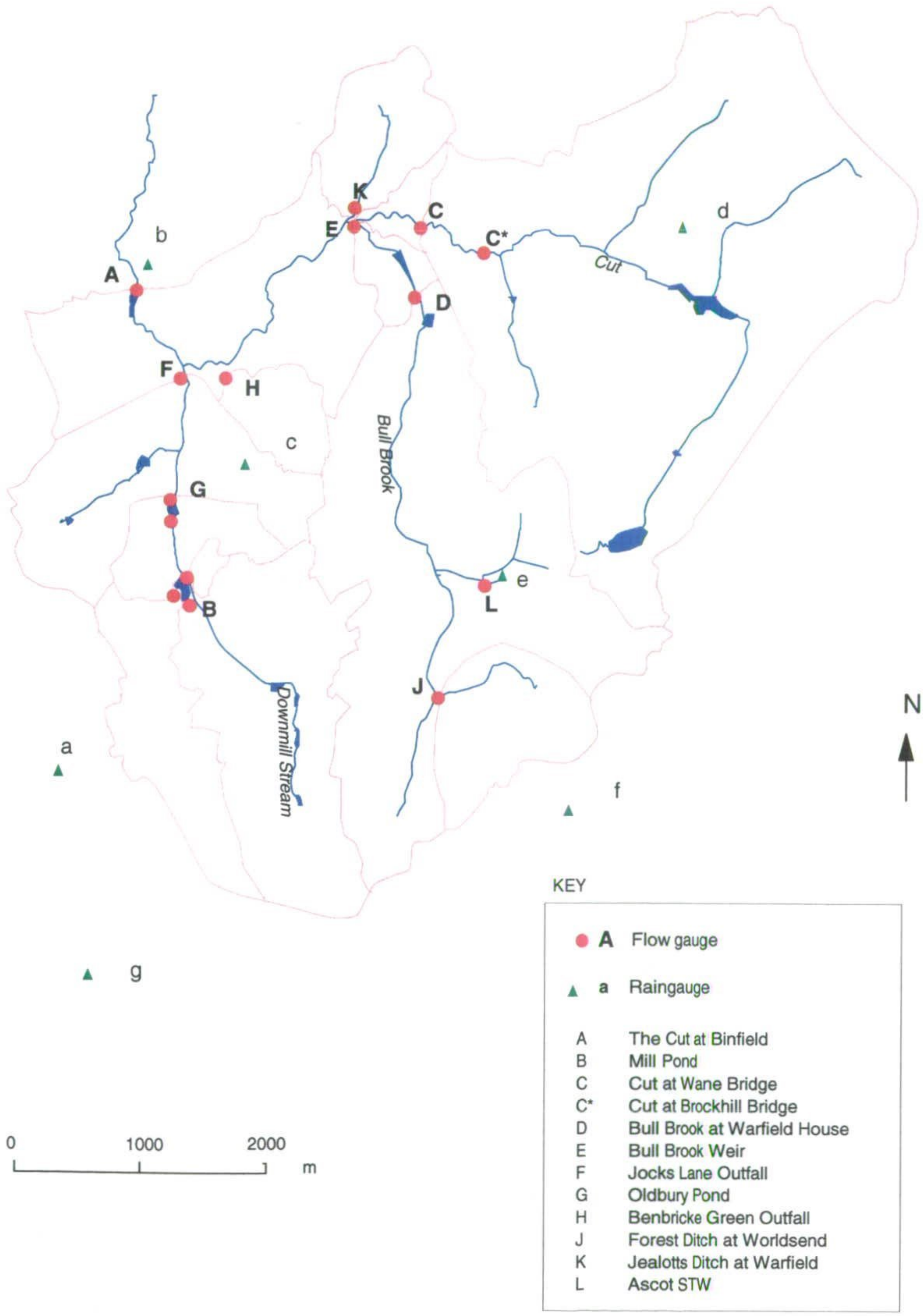


Figure 3.3

'unbalanced' urban runoff, and leaving only a few very minor storm sewer outfalls to the Cut unmonitored.

- to monitor runoff from single land use rural areas to compare with the urban residential, industrial and commercial areas. The forest catchment at Worldsend (J) and the pasture catchment herein called Jealotts Ditch (K) were chosen.
- to monitor rainfall at sufficient locations to define areal storm profiles. The existing Met.Office and EA recording gauges at Beaufort Park/Easthamstead (a) and Bracknell Sewage Treatment Works (b) were supplemented by four new recording raingauges at Bracknell Town Centre/3M (c), Winkfield (d), Ascot Sewage Treatment Works (e), and Berkshire Golf Course (f). Additional daily rainfall data were obtained from the Met Office gauge at Broadmoor (g).
- to obtain daily Soil Moisture Deficit data for the Beaufort Park meteorological site.
- to establish 'dry weather' and storm flow discharges from Ascot Sewage Works.

Figure 3.3 shows the location of all the flow gauges and raingauges used in this study, together with the subcatchment boundaries for each flowgauge (defined as described in Appendix B using the IH-Digital Terrain Model and BFBC's sewer layout database). The same subcatchment boundaries were also shown (in white) on Fig. 3.1 to indicate land use, which is summarised in Table 3.2 below.

Table 3.2 Area and land-use of gauged catchments.

Ref	Gauge Name	Area (km^2)	Farm/ Grassland %	Wood/ Heath %	Urban-Residential %	Industrial %	Commercial %	Lakes %	Rural Residential %	URBAN (FSR)
A	The Cut at Binfield	51.91	46.0	21.6	21.0	2.9	1.3	0.6	6.6	0.285
B_1	Mill Pond, Easthampstead Inlet	5.17	17.2	30.2	52.2	0	0	0.4	0	0.522
B_2	Mill Pond, Great Hollands Inlet	2.29	11.2	31.3	57.5	0	0	0	0	0.575
B_3	Mill Pond, Wildridings Inlet	0.14	15.6	0	84.4	0	0	0	0	0.844
B_4	Mill Pond Outfall (at Oldbury)	7.66	15.7	29.7	54.0	0	0	0.6	0	0.540
C	Cut at Wane Bridge	18.48	70.1	14.0	0.5	0	0	0.8	14.6	0.078
D	Bull Brook at Warfield House	12.29	12.9	48.0	36.3	1.5	0.5	0.3	0.5	0.386
E	Bull Brook Weir	12.87	15.9	46.2	34.6	1.4	0.5	0.4	1.0	0.370
F	Jocks Lane Outfall	12.55	21.6	19.8	42.4	10.6	3.5	0.6	1.5	0.573
G_1	Oldbury Pond, Industrial Inlet	0.62	34.0	0	40.7	18.5	6.6	0.2	0	0.658
G_2	Oldbury Pond, Waitrose Inlet	0.51	0.7	0.9	0	98.4	0	0	0	0.984
G_3	Oldbury Pond, Bypass Outfall	1.13	18.9	0.4	22.3	54.7	3.6	0.1	0	0.806
H	Benbricke Green Outfall	1.13	23.5	0	62.2	0	13.6	0.7	0	0.765
J	Forest Ditch at Worldsend	2.04	0	99.8	0	0	0	0.2	0	0
K	Jealotts Ditch at Warfield	1.62	97.5	1.9	0	0	0	0	0.6	0.003

In Table 3.2, the overall percentage of URBAN land-use has been taken as the sum of: urban-residential + commercial + industrial + half the rural-residential land-use. Note that gauge L at the Ascot Sewage Treatment Works is not included in this table since its catchment area was undefined and its impact expected to be small. Note also that the original choice of off-line pond to monitor was Bay Road (Table 3.1, no.5), but with three inlet pipes at the upstream end of the side weir channel, and one part way along, it would have required too many monitors and also would probably have had an unstable flow profile alongside the side weir. Finally note that only 14% of the Benbricke Green catchment is 'commercial'; originally another monitor was planned between the commercial and residential areas, but potential manholes for its installation were either inaccessible or hydraulically unsuitable.

Further details of the gauges and data processing, together with various plans and photographs, are given in Appendix B.

3.3 DISCUSSION ISSUES RELATED TO MONITORING STRATEGY

3.3.1 Binfield, level recorders, historic data and rating equations

At the start of the project, the EA flow data for the Cut at Binfield (A), and the temporary level recorders (C,D,E) were obtained in virtually continuous form, back to the start of the 15-minute data (1986/8). To speed up transfer times, these large amounts of data were obtained in Hewlett Packard unformatted binary files, requiring the development of suitable read and display software at IH. However, upgrading to the EA archive facilities meant subsequent data could be obtained as ASCII files.

Initial analysis concentrated on identifying trends in the historic data. The Binfield data showed significant increases in flood discharges over the urbanising period (mean annual flood changing from 6.55 to 10.25 m^3/s, see Figs 3.4 and 3.5) and a significant trend towards summer flood maxima (the percentage of all floods over 6 m^3/s that occurred between May and October increasing from 32% to 54%, see Fig. 3.6a and b). Thus, despite the storage ponds provided in Bracknell, mean annual flood appears to have risen by 56%. The number of floods above 6 m^3/s has increased by 72% and there has been a significant change in the season that floods occur. Analysis of the temporary gauges (C,D,E) appeared to exhibit drifting zero levels, possibly caused by silting in the channels or stilling wells, or by instrument drift. As this drift could not be corrected with confidence the historic data has not been analysed further.

The EA continued data collection at Binfield and the temporary level recorders, and in June 1993 they installed new level recorders on the three Mill Pond inlets (B). All data were collected at the standard 15-minute timestep, though it was realised this was too long to record accurately the rapid response at the Mill Pond. The new level recorder at Wildridings was vandalised in early summer 1995, and not replaced until summer 1996, when at the EA's request, the full field programme (except Binfield) was taken over by IH. At that time, the timestep at the Mill Pond and Warfield House recorders was changed to 5-minutes, and a level recorder was installed on the outlet weir from Ascot STW. Theoretical rating curves were developed at IH for the Mill Pond flumes and the weir at Ascot STW; the development of ratings for Warfield House and Wane Bridge is discussed later.

Annual and Monthly Flow Maxima at Binfield

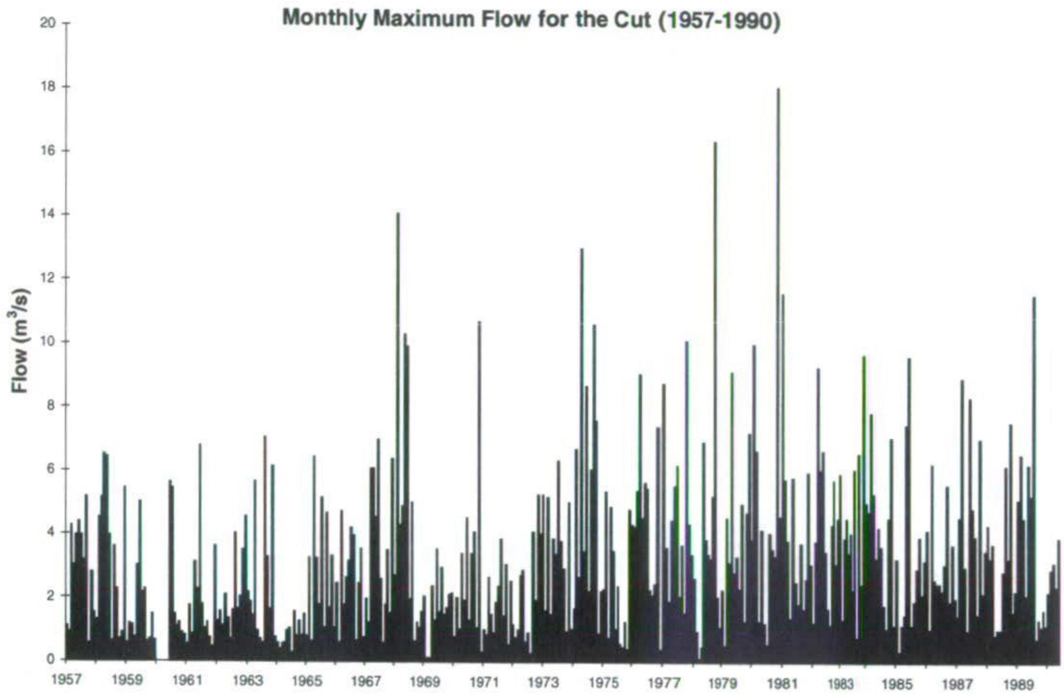


Figure 3.4

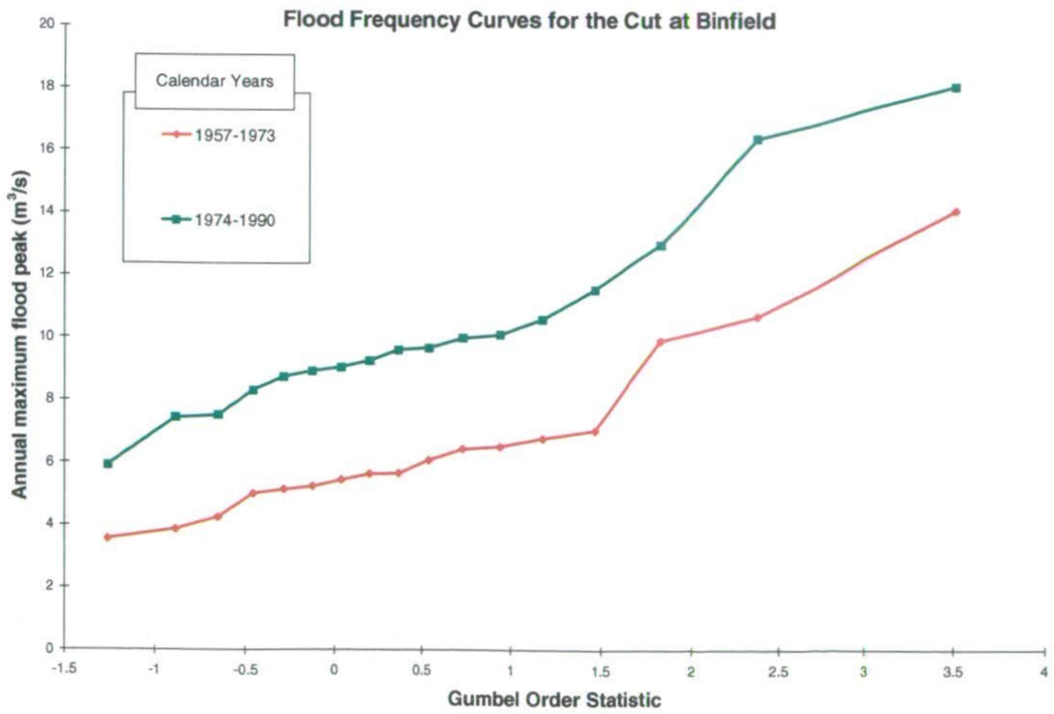
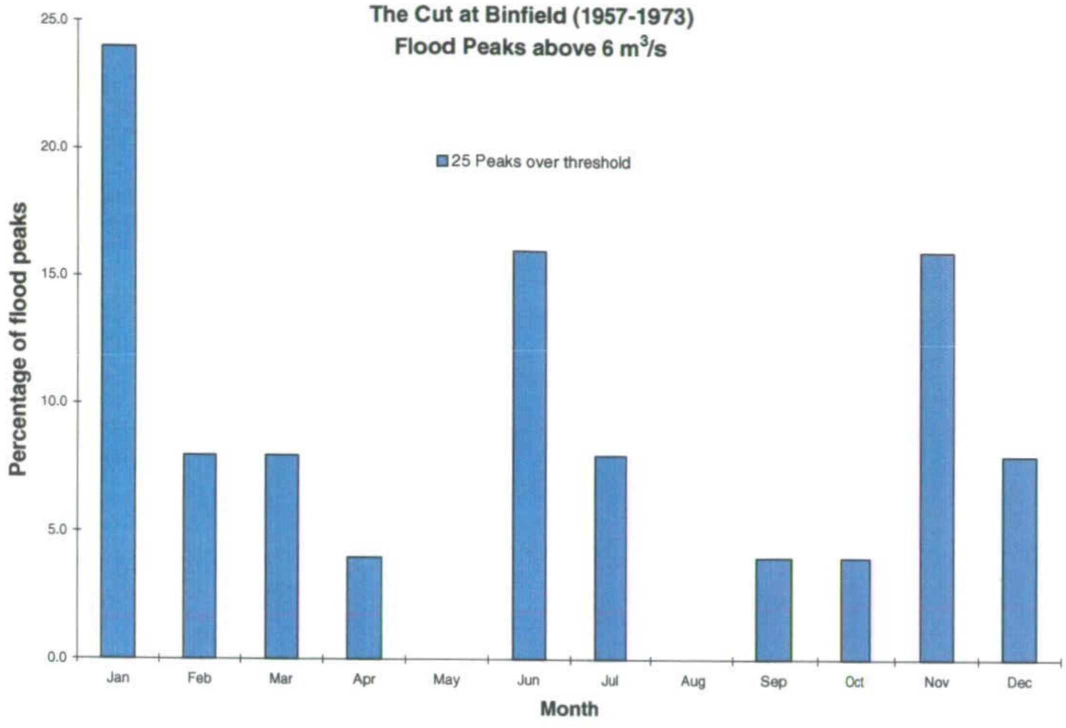


Figure 3.5

Changing Monthly Floods with Urbanisation



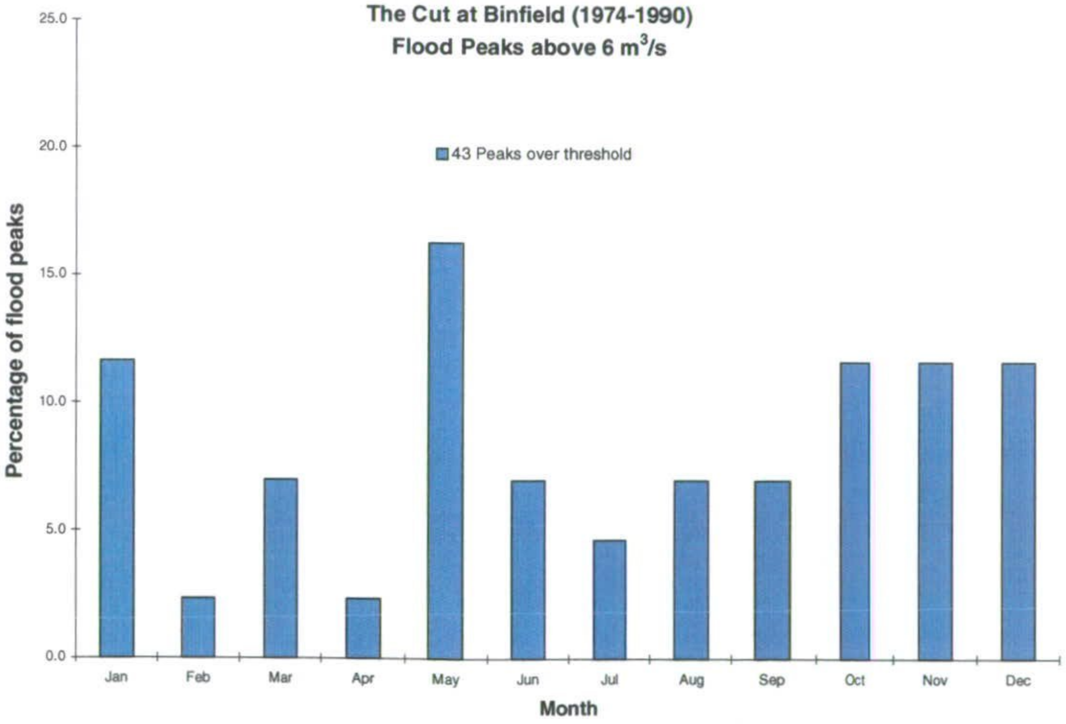


Figure 3.6

3.3.2 Depth/velocity monitors and Jocks Lane

Apart from the level data collected as described above, all the flow related data collection (Table 3.2, sites B_4 to K excluding E) was based on depth/velocity monitors similar to those used by sewer survey contractors. These monitors can be installed on a temporary or semi-permanent basis, and by measuring flow depth and velocity avoid the need to construct permanent controls with fixed depth-discharge relationships. Four different makes of monitor have been used (ADS, Detectronic/Montec, Prolec, and Unidata), each with their own data retrieval and archiving software procedures. Each make used an ultrasonic Doppler device to measure flow velocity (calculated as a point/regional velocity rather than a profile average), and each used a pressure transducer to measure depth (though the ADS monitors use a downward seeking ultrasonic gauge under normal free surface flow - i.e. unless it is surcharged pipe flow).

The ADS monitors were chosen for Jocks Lane (F) on the advice of IHS, the sewer survey/modelling contractors, now independent, but then a subsidiary of HR Wallingford. The Jocks Lane outfall is not a single culvert, but three separate 1800 mm pipes. These pipes are cross connected, but drain separate source catchments and thus needed to be monitored separately. The ADS system, developed in the USA, was thought better suited to long-term gauging in larger pipes. At the time ADS would only make the system available as a fully managed package, but, wanting a suitable demonstration project, they offered the package at a large discount. The monitors were installed in March 1993. Data at a 5-minute timestep was telemetered to the ADS offices for processing, and supplied to IH in ASCII form at approximately six month intervals. ADS have subsequently withdrawn from Europe, but the monitoring continued via a franchise arrangement between ADS and IHS.

Evaluating the Jocks Lane discharge from three monitors obviously compounds measurement errors and increases susceptibility to instrument malfunction. Although cross connected, the pipes do not respond in phase, and flow in each pipe changes very quickly (from 20 to 2000 l/s in a single timestep on some occasions). It has not therefore been possible to assess errors or to fill missing data by cross correlation. The ADS processing system does allow suspect/missing velocity measurements to be replaced with estimates derived from a Manning equation, but it was not always clear when this was done or what Manning 'n' was used (see Appendix B). This concern was heightened by a tendency at one of the monitors for peak velocity to lag behind peak depth, and at another monitor for velocity to increase in falling flow. Such effects could be due to sediment impacts on the Doppler velocity measurement, but are more probably caused by changes in flow profile (i.e. the flow is not at 'normal depth'), in which case using a fixed Manning 'n' to infill data would not be fully justified. Despite these concerns and a long period when one velocity sensor was not operational, a virtually continuous flow record has been derived for this site.

3.3.3 Detectronic monitors at Oldbury, Mill Pond Outfall and Benbricke Green

Detectronic/Montec monitors (favoured by most sewer survey contractors) were used at the Oldbury storage pond (G), on the Mill Pond outlet (B), and at Benbricke Green (H). They were installed in June 1993. These monitors were managed for most of the study by the EA, and the data were transferred to IH at approximately 6 month intervals. Data were collected at a basic 30 minute timestep, changing automatically to 5 minutes when flow depth exceeded 100 mm. Some 'hunting' of the timestep occurred, causing data processing problems (the Detectronic processing software stored successive data sequences at each timestep separately and could not print them out together).

The Oldbury storage pond (G) is an off-line 'dry pond', originally doubling as a horse paddock but subsequently re-developed as a wasteland under a 'car-park on stilts' (see Appendix B). The 'bypass' monitor, sited near the outfall from the 1050 mm throttle pipe, shows a fairly consistent depth-velocity relationship, but includes a change in flow conditions from sub to super and back to sub-critical between the depths of 100 and 300 mm. The monitors on the two 1300 mm inlets were affected by variable backwater from the throttle pipe and show a marked 'loop rating'. Thus all three monitor sites confirmed the need to monitor both depth and velocity; sites with more consistent ratings would have been preferred, but upstream manholes involved junctions, bends and access difficulties. The observed data provide scant evidence of flow into the pond at any time during the four year study period, though some scouring in the pond near the side weir suggested at least one such occurrence had occurred. In retrospect, a level gauge should have been included in the side weir channel, and probably also in the pond (it had never been expected to <u>fill</u>, and with twin 200 mm outlet pipes it was expected to yield a long draindown). The lack of observed spill into the pond meant it was not possible to assess off-line pond performance.

Alongside the outfall of the Oldbury bypass is the outfall of the 870 m long, 1350 mm diameter culvert from the Mill Pond (see Appendix B). This was chosen as the most convenient site to monitor the Mill Pond outflow (in preference to the 'drowned flume' where access was offset from a deep manhole, causing considerable safety concerns). This site showed, for the most part, a stable (subcritical) depth-discharge relationship, but included a few periods of low velocity readings (possibly due to 'ragging' of the Doppler sensor). Higher flows also seemed to be throttled, probably by the Mill Pond outlet control, but possibly also by a constriction within the culvert. Detailed examination of the sewer data has identified a change in culvert section (1350 mm to twin 900 mm pipes) at a road crossing, possibly involving a backfall. This was discovered late in the study, and needs to be confirmed in the field. In practice any effect is likely to be secondary to the main outlet throttle at the pond, and it has not been incorporated in the model studies of Chapter 5.

The Benbricke Green outfall (H) consists of twin 1050 mm pipes running in parallel from the last major confluence. Monitors were installed on both pipes in June 1993. As a relatively small, steep catchment with no storage ponds, the rapid changes in depth and velocity were not fully represented by the 5-minute timestep. Also there were the usual problems of 'ragging' of the velocity sensor, and periods of missing data from one monitor. Flow was not split equally between the pipes, and some preference of flow from one or other side of the upstream confluence was indicated. Cross correlation between the monitors was used to estimate missing depths, and a single depth:discharge relationship was determined by overlaying the data from each gauge (there was no backing up from the Cut, but a slight 'loop rating' effect was found).

3.3.4 Worldsend, Jealotts Ditch, and ratings for Wane Bridge and Warfield

Having instrumented the urban subcatchments, the strategy for monitoring the rural subcatchments, Worldsend (J) and Jealotts Ditch (K), was reconsidered. Originally it was intended to monitor just water level (by pressure transducer) and, as at Wane Bridge (C) and Warfield House (D), seek a rating through the short term deployment of flow monitors. As the respective drains were dry in June 1993, instrument installation was deferred. In October the streams were observed to 'back up' during a flood event, and it was clear both depth and velocity would need to be monitored. The additional budget to purchase flow monitors was not available until 1994, and further delays due to logistics, instrument unreliability, and gaining permission to install monitors in road culverts, meant the rural monitors were not installed until February 1995.

Prolec monitors (little changed from the original 1981 Golden River design) were selected through competitive tender, and were installed at Worldsend, Jealotts Ditch, and also to calibrate the Warfield House level recorder. However, they suffered vandalism and accident damage, and also proved rather unreliable. Two were replaced with newly available, less obtrusive, and much cheaper Unidata 'Starflow' monitors. A Starflow monitor was also installed at Brockhill Bridge, upstream of (but as a calibration for) Wane Bridge (where the section could not be rated with a flow monitor since it involved an upstream bend and a bridge pillar in mid channel causing the flow to split unevenly). Each monitor location was chosen in a prismatic channel (pipe or rectangular bridge opening) and where concealment for security was possible. The Bull Brook weir site (E) was not rated as it was silted and overgrown; at low flows the upstream flow seemed to be supercritical, and at high flows the weir could drown. The level only data have however allowed useful comparisons with Warfield House.

Very little reliable data were obtained from Worldsend and Jealotts Ditch. The channels were dry much of the year, and flows were comparatively low even in winter - suggesting a deeper sub-surface flow path from these areas to the river system. For Brockhill Bridge, the channel to Wane Bridge was subject to extensive weed growth in summer, and the travel time increased from about 50 minutes to 5 hours. Separate winter and summer ratings have been developed, though summer flow typically accounts for less than 15% of the volume at Binfield. A better rating equation was derived for Warfield House (see Appendix B), but there were extended periods (lasting months) when the section was drowned (possibly due to blockage of the channel within the private grounds of Warfield House, or to unspecified activities at the ornamental lake). Such periods could usually be identified, and at those times the rating was not applied.

3.3.5 Depth/velocity monitors, general opinions

It must be noted that the gauge locations used in this study were not ideal, and data reliability was not high (but flow measurement in small ephemeral streams is known to give difficulties). The flow monitors were originally developed for sewer use, where pipes may 'backup', but rarely run dry and flow profiles are fairly stable. The pressure transducers are subject to drift (a tolerance of 2-5 cm is not unusual), and Doppler velocity signals vary with sediment characteristics (an accuracy in flow measurement of ±25% is commonly accepted). They are least accurate at low depth and where velocity is unevenly distributed or pulsating (probable in wide channels). However, flow monitors do not restrict the flow, are relatively unobtrusive and thus less vandal-prone. The alternatives of weir or flume structures are more costly, raise upstream levels, need more maintenance and de-trashing, and tempt vandals.

With flow monitors it is fairly standard practice to plot 'scattergraphs' of velocity against depth to identify poor hydraulic conditions. Most of the 'noise' in the monitor comes from the velocity signal, but none of the manufacturers' software packages attempt any smoothing, by for example averaging implied values of Manning roughness (ADS do estimate velocity by a fixed Manning 'n' for checking and infilling missing data). In this study, software has been developed to apply a moving average to the implied Manning 'n' derived with any of the monitor types (see Appendix B).

Of the monitors maintained directly by IH, the Detectronic had cost approximately £4000 each, but were the easiest to use. The Prolec monitors had cost approximately £3000. The Unidata Starflow monitors cost approximately £1200, and were technically fine but required greater care in field use (some data periods were lost due to downloading/reprogramming accidents). A number of other makes of monitor exist. Electromagnetic gauges cost approximately £15,000 a site and require mains electricity. TWU have recently collaborated in developing a high accuracy Doppler monitor based on full velocity profiling (rather than a point/regional velocity); these monitors are likely to cost about £20,000. The monitors used represented a balance between cost and performance within the study aims and resources.

3.3.6 Rainfall, Soil Moisture, Dry Weather Flow, and concluding remarks

With respect to rainfall monitoring (point 8 in the strategy), the additional recording raingauges were installed between January and April 1993. Recording raingauge data for the Met.Office gauge at Beaufort Park/Easthamstead have not been obtained (for reasons of cost), but daily data have been obtained for this site, and also for Broadmoor (g on Fig. 3.3). Weather radar data from the Chenies site were briefly assessed, but has not been used in the modelling studies of this report.

Daily Soil Moisture Deficit (SMD) data have also been obtained for Beaufort Park, with values as calculated by the original Grindley (1967) model (to conform with the Flood Studies Report). This method is now obsolete (replaced by MORECS), and a few gaps existed in the Beaufort Park data. A version of the model, based on estimating daily evaporation from a sine wave, was used to infill these missing periods.

The final point in the monitoring strategy was to derive Dry Weather Flow profiles for the outflow from Ascot Sewage Treatment Works and thus determine any storm response (see Appendix B). The data have not been used in the model studies described in Chapter 5. However, the diurnal variation in Dry Weather flow is clearly visible at both Warfield House and Binfield Weir, and could be used to confirm flow times through the Bull Brook.

Concluding this discussion, it must be stated that although the data collected in this study contain a range of errors and uncertainties, they still represent a unique picture of runoff processes through a mixed urban/rural catchment. The accuracy of some of the data may be poor in volume terms, but they contain valuable information in terms of timing and general response shape. In the modelling studies described in Chapter 5 only the Binfield data have been used for volume analysis and fitting, yet the other data provide firm evidence of the SCHEME model's ability to predict response patterns within the catchment.

3.4 SELECTION OF STORM EVENTS

Figure 3.7 shows a plot of the data gathered during this study for the four principle gauges: Wane Bridge on the Cut upstream of Bracknell, Warfield House on the Bull Brook draining eastern Bracknell, Jocks Lane on the (piped) Downmill Stream, draining western Bracknell, and Binfield on the Cut downstream of Bracknell. Following on from the discussion of the previous section, the drifting zero at Wane Bridge and the drowned flow periods (winter 1994 and autumn 1996) at Warfield House can be clearly seen. Considering all monitors, the periods of data that were missing or needing manual correction were extensive. It was thus decided the data could not be processed as continuous records, but would be better treated as isolated events. Thirty one event periods have been selected (from April 1993 to April 1997), varying in duration between one and four days. Their locations within the records given in Fig. 3.7 are indicated by the dotted red lines. The events were not chosen in a wholly objective fashion. Firstly all events yielding more than 5 m^3/s at Binfield were chosen, then all significant events at the Oldbury bypass, and finally a number of additional events were chosen to ensure a reasonable distribution of events during the year.

Four years of data from principal gauges

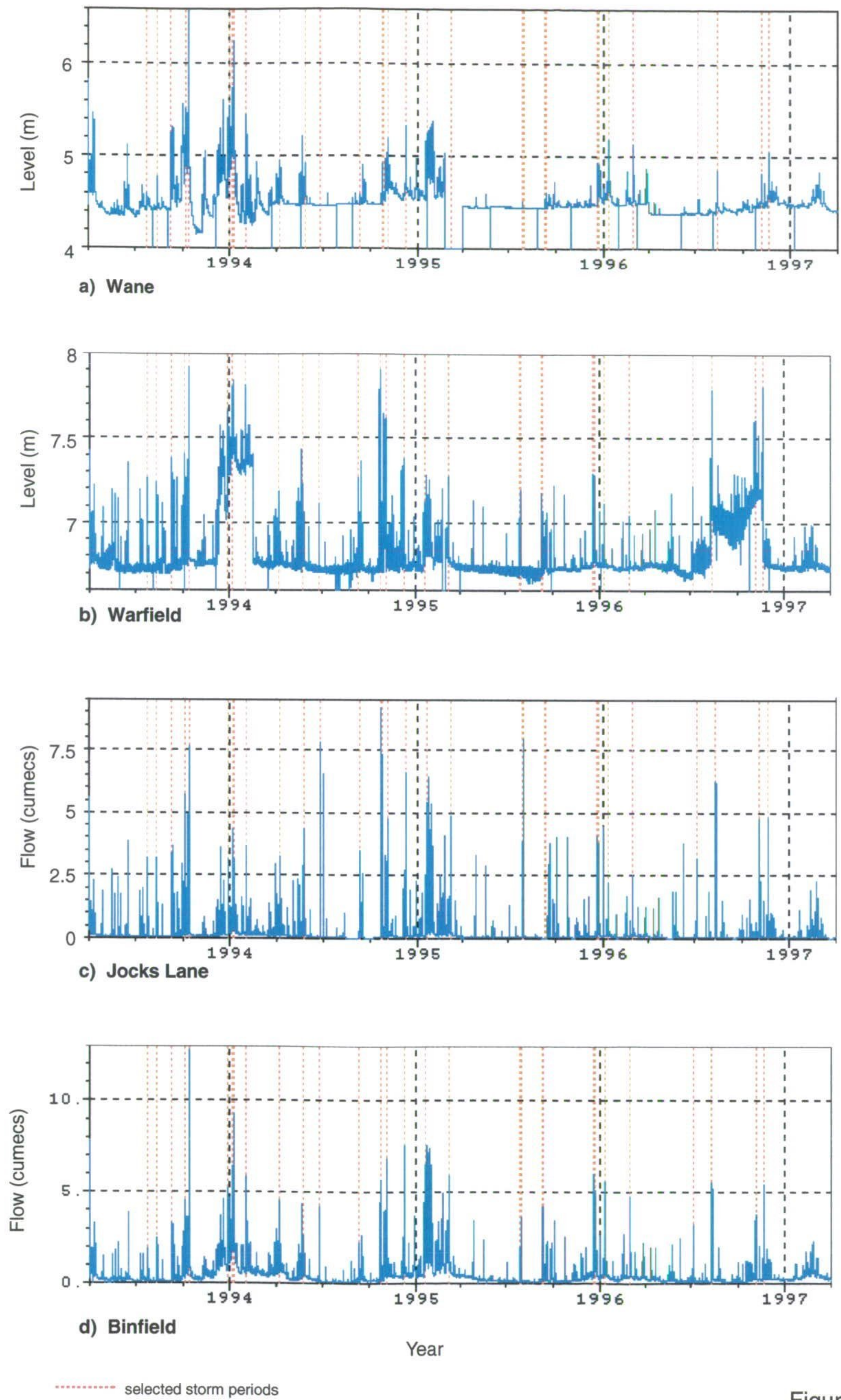


Figure 3.7

The selected events were extracted from the data series, and rating curves or velocity smoothing applied (following the procedures described in Appendix B). Table 3.3 gives the date, Soil Moisture Deficit, and maximum flow for each event at each of the main gauges (the letters in row one relate to their location on Fig. 3.3). For Worldsend, Jealotts ditch and Ascot STW, the late installation of gauges, long dry periods, and miscellaneous acts of vandalism have meant that data were available for very few of the selected events (7, 6 and 2 events respectively), and the data have mainly been used in visual assessment of fit during catchment modelling (see Chapter 5). Note that the early events at Warfield were based on the derived rating equation, but from event 20 onwards were taken directly from the Starflow data.

Table 3.3 Maximum flow (m^3/s) for each event and gauge

			C	D	H	F	A	B_1	B_2	B_3	B_4	G_1	G_2	G_3
No	Date	SMD mm	Wane Bridge	War-field	Ben-bricke	Jocks Lane	Bin-field	E.ham-pstead	G.Hol-lands	Wild-ridings	Mill-Pond	Indus-trial	Wait-rose	Oldbury Bypass
1	23-7-93	127.8	0.06	2.68	0.81	3.17	1.93	1.20	0.76	0.20	0.51	0.57	0.37	1.31
2	12-8-93	134.7	0.25	2.49	0.35	3.18	2.44	1.38	0.80	0.15	0.76	0.54	0.33	0.86
3	9-9-93	124.3	0.14	1.67	0.44	2.72	1.58	1.24	0.79	0.20	0.52	0.63	0.34	0.96
4	6-10-93	48.6	2.41	3.75	0.74	5.72	4.53	3.21	1.22	0.25	1.20	0.24	0.53	1.40
5	12-10-93	35.1	10.35	8.61	0.80	7.61	12.77	3.32	1.63	0.19	1.43	0.25	0.77	1.47
6	30-12-93	0	4.43		0.31	3.16	5.18		0.61	0.06	0.90	0.16	0.08	0.60
7	6-1-94	0.0	5.91		0.36	4.30	6.44		0.72	0.10	1.45	0.13	0.16	0.63
8	8-1-94	0	10.60		0.35	3.11	9.31		0.49	0.06	1.13	0.08	0.07	0.46
9	3-2-94	0.0	3.76		0.45	3.69	5.83		0.64		1.30	0.09	0.14	0.64
10	8-4-94	0	1.15	2.17	0.41	3.23	4.51	1.57	0.99	0.05		0.29	0.18	0.58
11	25-5-94	6.9	1.04	2.45	0.67	4.35	2.63	1.12	0.50		0.94	0.45	0.21	0.96
12	24-6-94	59.6	0.03	1.74	0.56	7.82	4.16	2.22	4.19		1.36	2.00	0.46	1.44
13	9-9-94	124.2	0.05	2.71	0.46	3.50	2.23	1.85	0.79		0.76	0.68	0.16	1.22
14	22-10-94	111.3	1.06	8.44	0.61	9.20	5.64	3.36	1.37		1.35	0.74	0.41	1.48
15	25-10-94	94.1	0.22	2.71	0.67	7.35	2.64	0.82	0.81		0.31	0.98	0.51	1.35
16	4-11-94	49.3	2.27	5.59	0.42	4.78	6.83	1.38	0.64		0.11	0.39	0.26	0.76
17	8-12-94	17.0	3.04	3.53	0.73	6.64	7.53	2.24	1.42		1.71	0.79	0.66	1.41
18	19-1-95	0	2.51	1.77	0.72	5.40	6.48	1.20	0.59		1.05	1.20	0.50	1.45
19	7-3-95	1.5		2.73	0.38	4.88	5.92	1.74	0.85		1.39	0.43	0.39	0.88
20	26-7-95	133.2	0.02	1.52	0.55	3.84	1.96	1.93			1.09	0.50	0.41	1.09
21	27-7-95	132.1	0.02	2.66	0.77	7.96	3.58	1.10			0.42	0.59	0.61	1.21
22	7-9-95	135.7	0.05	2.43	0.40		4.19	1.25			0.96	0.63	0.43	1.07
23	10-9-95	126.1	0.07	1.63	0.52		4.23	1.02			1.07	0.97	0.52	1.33
24	19-12-95	54.5	1.11	3.20	0.32	4.11	6.01	1.65			1.17	0.41	0.38	0.77
25	21-12-95	33.0	1.06	3.26	0.34	3.87	5.08	2.10			1.26	0.57	0.39	0.90
26	8-1-96	16.9	2.22	1.96	0.31	2.24	5.58	0.74	0.38			0.25	0.10	0.52
27	24-2-96	4.0	1.97	1.73		2.53	4.74	1.00	0.64			0.41	0.25	0.69
28	5-7-96	106.4	0.05	1.36		3.18	3.24	1.05	0.66			0.45	0.36	0.93
29	9-8-96	129.3	0.36	2.78		6.27	5.51	4.74	2.55			2.11	1.40	1.45
30	3-11-96	125.8	0.69	2.04		4.76	3.75	1.57	0.72	0.12	0.60	0.49	0.34	0.83
31	16-11-96	93.6	1.60	5.00		4.81	5.45	1.67	0.80	0.13	0.46	0.19	0.34	1.03

To provide a broad assessment of the quality of the data, the total runoff volume passing each gauge during each event was also determined. This is given in Table 3.4 along with a percentage column comparing inflow to outflow volumes for specific reaches and storage ponds. The first percentage column compares the tributary inflows to the Cut between Wane Bridge and Binfield with the volume at Binfield. The second and third percentage columns compare the inflow and outflow volumes at the Mill Pond and Oldbury pond. Despite great effort, considerable discrepancies persist in the data, as shown by the deviations from 100%. Note however that the figures are for events and ignore the effects of any continuing recessions. Note also that Benbricke and Wildridings data form a relatively minor component of their respective percentage columns, and their periods of missing data do not unduly affect this preliminary assessment of the data.

Table 3.4 Volumes (1000 m^3) during event period for each event and gauge

No	C Wane Bridge	D War-field	H Ben-bricke	F Jocks Lane	A Bin-field	% of A	B_1 E.ham-pstead	B_2 G.Hol-lands	B_3 Wild-ridings	B_4 Mill-Pond	% of B_4	G_1 Indus-trial	G_2 Walt-rose	G_3 Oldbury Bypass	% of G_3
1	12.2	91.7	1.4	39.8	75.0	193	16.9	2.8	1.3	5.8	360	1.5	0.8	3.4	67
2	13.8	67.3	2.4	44.7	72.7	176	17.5	4.8	0.8	10.7	216	1.8	0.9	5.5	50
3	6.4	38.1	1.1	19.5	28.8	226	8.1	1.7	0.3	4.6	219	1.2	0.5	2.4	66
4	157.8	91.1	2.5	55.5	139.2	220	20.1	4.8	0.7	10.5	244	0.4	0.9	4.6	28
5	1037.5	528.3	18.3	307.2	1155.1	164	95.1	29.9	3.3	68.7	187	2.7	7.8	29.9	35
6	367.6		7.1	104.8	465.2	103		27.5	2.4	28.6	105	0.9	0.6	10.6	14
7	579.4		8.2	119.1	571.5	124		16.5	2.1	30.5	61	0.8	0.8	10.4	15
8	1209.2		15.5	212.4	1129.7	127		33.7	3.2	60.4	61	1.1	1.0	14.7	14
9	189.8		7.1	111.1	455.4	68		21.5		28.0	77	0.5	0.8	9.2	15
10	155.8	162.5	6.4	97.5	480.7	88	21.6	16.0	0.3			2.5	0.7	9.0	35
11	125.8	160.7	1.6	78.2	251.1	146	16.1	16.2		43.0	75	0.8	0.4	1.9	62
12	1.3	28.1	2.0	49.6	59.1	137	10.4	10.8		11.6	182	3.3	0.5	4.6	83
13	7.4	59.0	1.0	29.9	59.5	163	5.6	3.4		6.2	145	1.1	0.2	2.3	55
14	49.0	186.4	3.1	71.6	122.2	254	17.6	6.7		22.8	107	3.0	1.8	6.5	73
15	32.4	119.2	1.9	44.6	89.6	221	6.8	3.0		0.9	1117	0.9	1.0	4.0	45
16	235.4	362.4	9.8	167.4	474.4	163	35.2	16.8		1.6	3229	7.0	4.2	16.6	67
17	295.1	154.4	9.9	158.4	589.0	105	27.5	14.6		42.3	100	4.7	3.4	15.3	52
18	250.9	131.7	10.1	161.4	502.3	110	22.2	9.8		39.4	81	5.4	3.4	13.3	66
19		139.7	5.6	108.7	439.6	58	19.5	5.5		31.6	79	2.8	2.6	8.9	60
20	2.1	20.2	1.1	19.6	29.9	144	4.0			8.7	45	0.7	0.7	2.0	68
21	1.6	20.8	1.3	22.2	35.6	129	2.1			4.1	52	0.6	0.8	2.0	69
22	7.8	22.1	1.7		85.8	37	5.7			8.1	70	2.2	1.4	4.3	83
23	11.1	51.5	4.5		175.5	38	15.6			23.4	67	4.6	2.3	8.6	80
24	64.5	122.4	7.6	124.1	293.5	109	36.1			38.0	95	5.4	5.0	15.5	67
25	186.3	218.3	9.7	201.7	592.6	104	53.9			58.1	93	6.6	6.7	21.9	60
26	215.9	182.6	14.0	143.9	531.1	105	39.9	20.0				6.4	4.2	16.8	63
27	240.5	169.9		157.9	666.8	85	26.5	21.0				6.2	5.7	16.5	72
28	5.8	64.6		50.0	105.4	114	23.8	7.0				2.1	1.6	6.2	59
29	27.1	160.0		141.2	320.3	103	291.3	26.5				9.4	5.4	17.9	83
30	61.5	89.9		66.1	165.0	132	42.2	12.7	1.4	5.3	1060	2.1	2.1	7.7	55
31	202.6	711.4		218.0	540.9	209	119.1	28.3	4.6	10.4	1463	2.7	5.1	22.1	35

At Binfield, when account is taken of missing data, volumes from storm 10 onwards appear to sum relatively close to 100%, giving some confidence in use of the data for whole catchment modelling. The earlier storms however consistently add up to more than 100%, which seems largely due to overestimation of flow at Wane Bridge. At the Mill Pond, the 15-minute data interval often misses the peak flows. For the first 9 storms total inflow appears to average about double the outflow, but thereafter seems to average close to 100%, except for obvious occasions when the outflow gauge was under-recording (see Appendix B). At Oldbury, the sum of the inflows has never exceeded 100% and is often below 50% of the bypass, though throttling to about 1.45 m^3/s has occurred several times and the side weir has also overtopped on occasions (scour seen on the downstream side). The inlet data are thus thought to be the more suspect. In any case, the data at neither storage pond are considered suitable for detailed study of pond performance, though they may still be used to assess model performance in the catchment as a whole.

It may be noted from Table 3.4 that the two largest volume events each yielded about 1,100,000 m^3 at Binfield, which may be compared with the approximate total available volume in flood storage ponds and lakes of 210,000 m^3. How much of this flood storage was mobilised during those events is a matter of some interest.

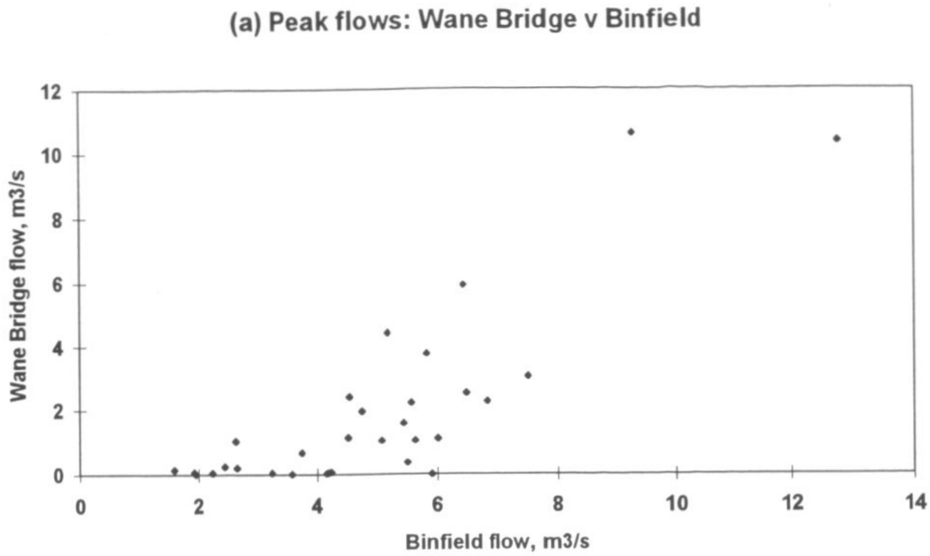
3.5 COMPARISON OF FLOOD RESPONSE BETWEEN GAUGES

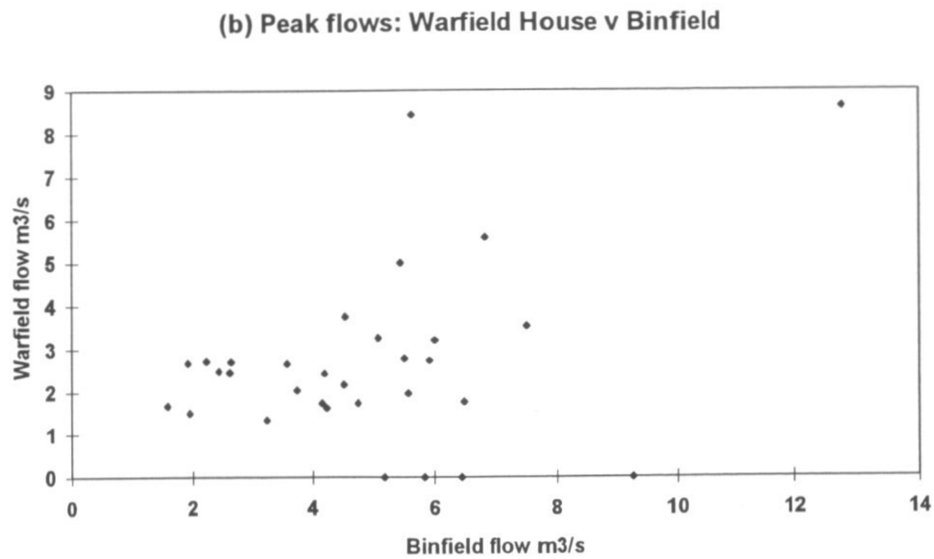
Some broad comparisons of flood performance between rural and urban catchments have been made using the data in Tables 3.3 and 3.4. Considering the four biggest flood peaks in the four years of data, at Wane Bridge (8% urbanised) these occur in January (2), December and October; at Warfield House (39% urbanised) they occur in October (2) and November (2), and at Jocks Lane (57% urbanised) they occur in October (2), June and July. Similar shifts to summer flooding in urban subcatchments can be seen throughout the data.

Lack of correspondence between the maximum flood periods in different subcatchments is further demonstrated by Fig. 3.8, comparing the peak discharge in each event at Wane Bridge, Warfield House and Jocks Lane with the peak from the same event at Binfield. Although the scatter is large (and bearing in mind the existing uncertainties in the data), Fig. 3.8a clearly shows a trend between the occurrence of peak flows at Wane Bridge and Binfield. The trend is less clear in Fig. 3.8b between Warfield and Binfield, and between Jocks Lane and Binfield (3.8c) there is no trend. More specifically, the largest event at Binfield is only the fourth largest at Jocks Lane; while the largest Jocks Lane event was only the tenth largest at Binfield. Although this lack of correlation is not surprising, it provides firm evidence that while peak flows in the rural Wane Bridge catchment may occur under the same storms conditions as at Binfield, peak flows in the urban Jocks Lane catchment come from different storm conditions. Based on the 4-years of data, it can be stated that the T-year floods from urban areas do not coincide with the T-year floods from rural areas.

This observation is reinforced by Fig. 3.9a which directly compares peak flow at rural Wane Bridge with urban Jocks Lane, showing no correlation. Figure 3.9b compares part urbanised Warfield House with Jocks Lane, showing weak correlation. Figures 3.9c and d do however show a generally stronger correlation between flood volumes (dependent more on rainfall depth and less on hydraulic routing).

The lack of correspondence between flood peaks is also shown by the smaller catchments. Figure 3.10 compares the fast responding catchments of Benbricke Green, Mill Pond Outflow and Oldbury with Jocks Lane. The small catchments are all highly urbanised, but could be


(a) Peak flows: Wane Bridge v Binfield
Wane Bridge flow, m3/s
Binfield flow, m3/s
0
2
4
6
8
10
12
14


(b) Peak flows: Warfield House v Binfield
Warfield flow m3/s
Binfield flow m3/s
0
1
2
3
4
5
6
7
8
9
10
12
14

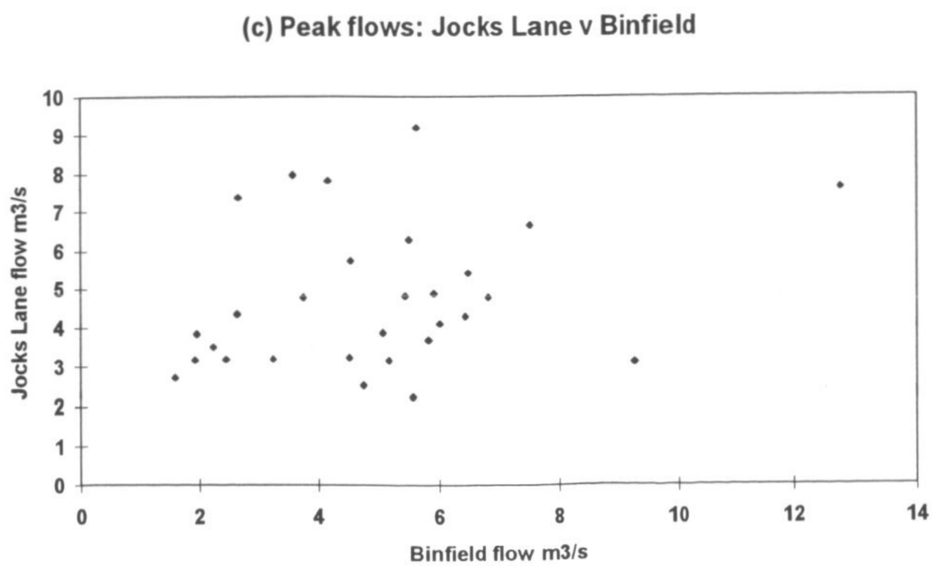

(c) Peak flows: Jocks Lane v Binfield
Jocks Lane flow m3/s
Binfield flow m3/s
0
1
2
3
4
5
6
7
8
9
10
12
14

Figure 3.8

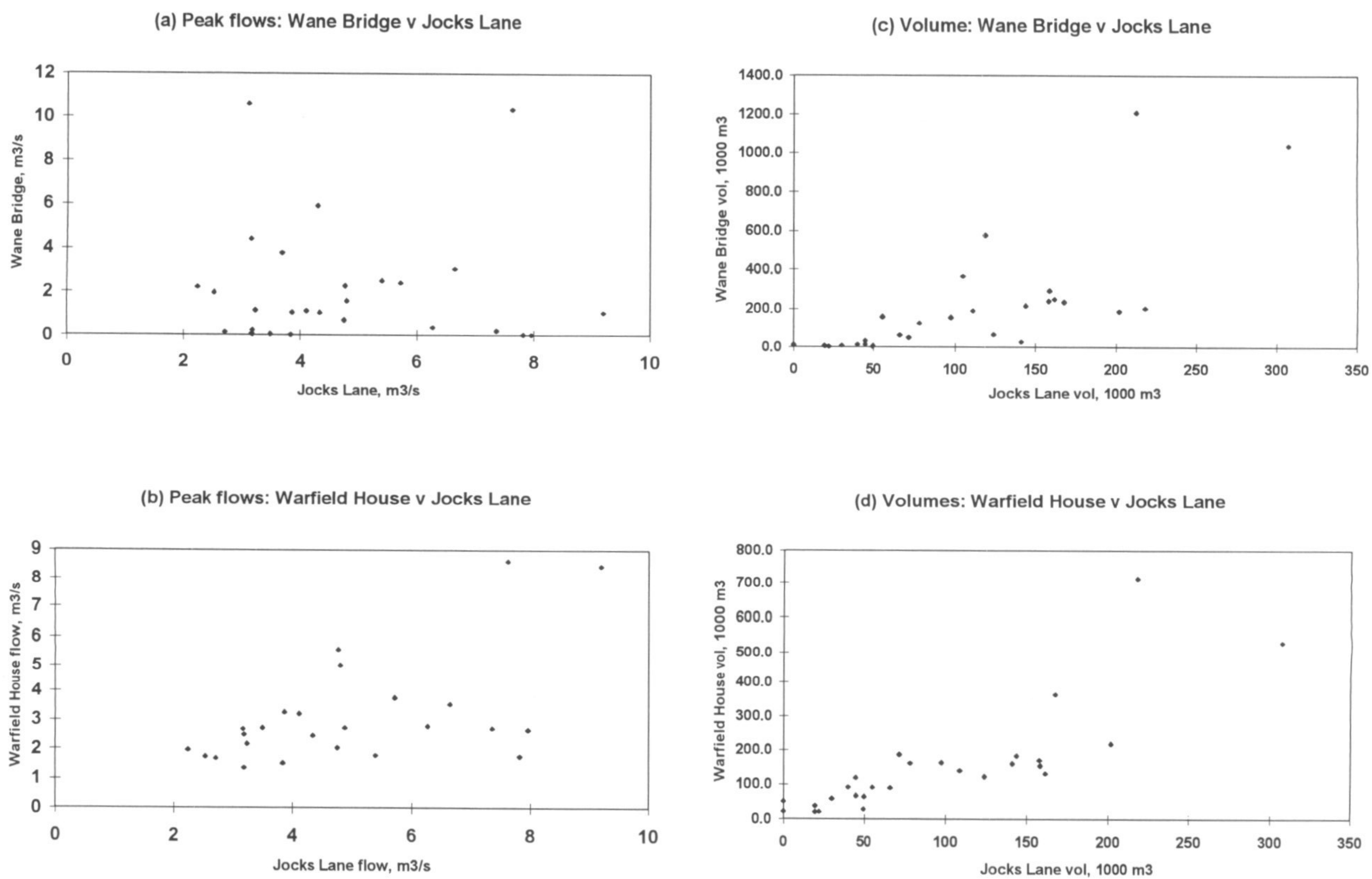

(a) Peak flows: Wane Bridge v Jocks Lane
Wane Bridge, m3/s
Jocks Lane, m3/s
(c) Volume: Wane Bridge v Jocks Lane
Wane Bridge vol, 1000 m3
Jocks Lane vol, 1000 m3
(b) Peak flows: Warfield House v Jocks Lane
Warfield House flow, m3/s
Jocks Lane flow, m3/s
(d) Volumes: Warfield House v Jocks Lane
Warfield House vol, 1000 m3
Jocks Lane vol, 1000 m3

Figure 3.9

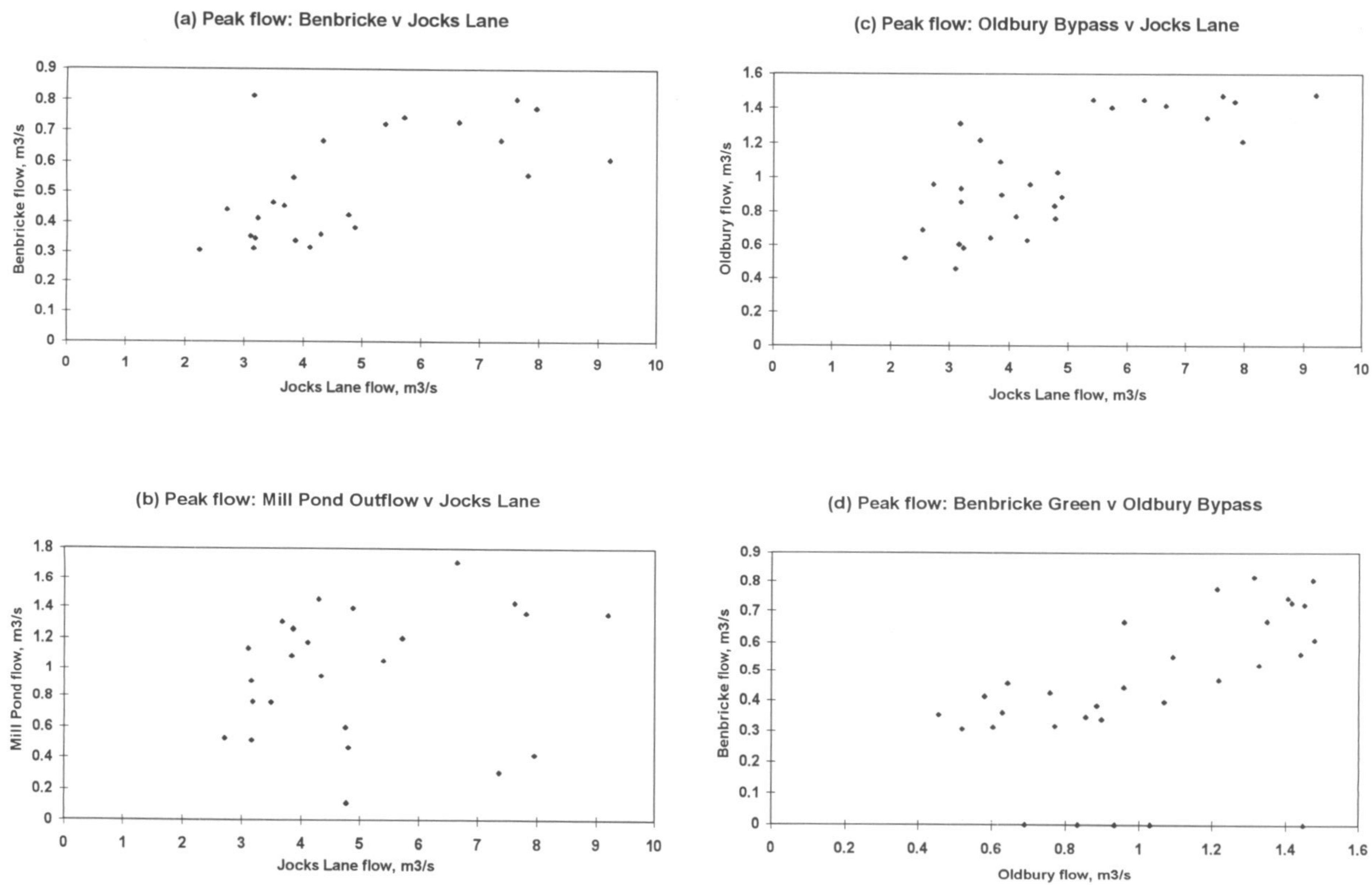

(a) Peak flow: Benbricke v Jocks Lane
Benbricke flow, m3/s
Jocks Lane flow, m3/s
(c) Peak flow: Oldbury Bypass v Jocks Lane
Oldbury flow, m3/s
Jocks Lane flow, m3/s
(b) Peak flow: Mill Pond Outflow v Jocks Lane
Mill Pond flow, m3/s
Jocks Lane flow, m3/s
(d) Peak flow: Benbricke Green v Oldbury Bypass
Benbricke flow, m3/s
Oldbury flow, m3/s

Figure 3.10

classified respectively as of largely commercial, residential and industrial land use. Their correlation with Jocks Lane is only slight - though for the Mill Pond(b) four of the five points below the main trend do coincide with periods when the flow monitor appeared to under-record velocity (see Appendix B). By comparison, a tighter correlation exists in Fig. 3.10d plotting Benbricke Green directly against Oldbury. Note also that the three smaller catchments all seem to exhibit a 'throttled' maximum (Mill Pond and Oldbury are both downstream of storage ponds).

Comparing flood volumes at the same three sites (Fig. 3.11) shows, as before, tighter correlation (and clearly shows the four Mill Pond points below the main trend). The consistency of these plots suggests that, despite specific concerns noted elsewhere, in general terms the broad quality of the data is acceptable. By excluding the few 'outlier' events a more reliable data set might yet be obtained.

The final plots presented here (Fig. 3.12) seek to explain the poor correlation between event peaks at Wane Bridge and Jocks Lane by their sensitivity to antecedent conditions. Flow peaks and volumes, as a percentage of the values at Binfield, are plotted against Soil Moisture Deficit. As might be expected, the plots show flow from the mainly rural Wane Bridge catchment becoming a smaller part of Binfield flow as Soil Moisture Deficit increases, while flow from the urban Jocks Lane catchment becomes more significant.

Figures 3.8 to 3.12 provide interesting comparisons between the flood response of different subcatchments and land uses, and are also useful as a means of identifying possible outliers in the data. However, the main thrust of this study was to calibrate and verify the rainfall-runoff model SCHEME. To this end the events have been split into 'test' and 'fit' sets for modelling, as described in Chapter 5.

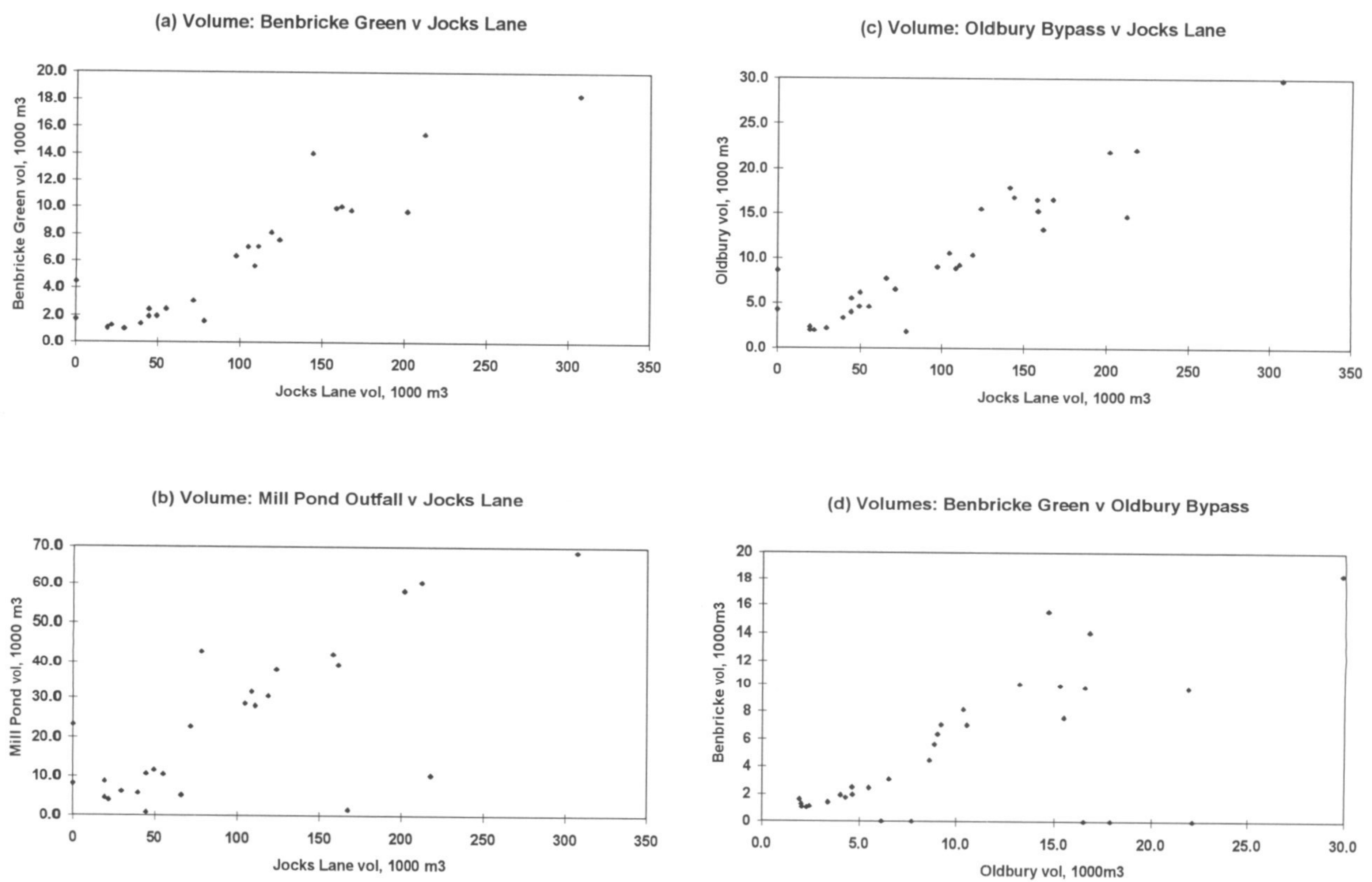

(a) Volume: Benbricke Green v Jocks Lane
Benbricke Green vol, 1000 m3
Jocks Lane vol, 1000 m3
(c) Volume: Oldbury Bypass v Jocks Lane
Oldbury vol, 1000 m3
Jocks Lane vol, 1000 m3
(b) Volume: Mill Pond Outfall v Jocks Lane
Mill Pond vol, 1000 m3
Jocks Lane vol, 1000 m3
(d) Volumes: Benbricke Green v Oldbury Bypass
Benbricke vol, 1000m3
Oldbury vol, 1000m3

Figure 3.11

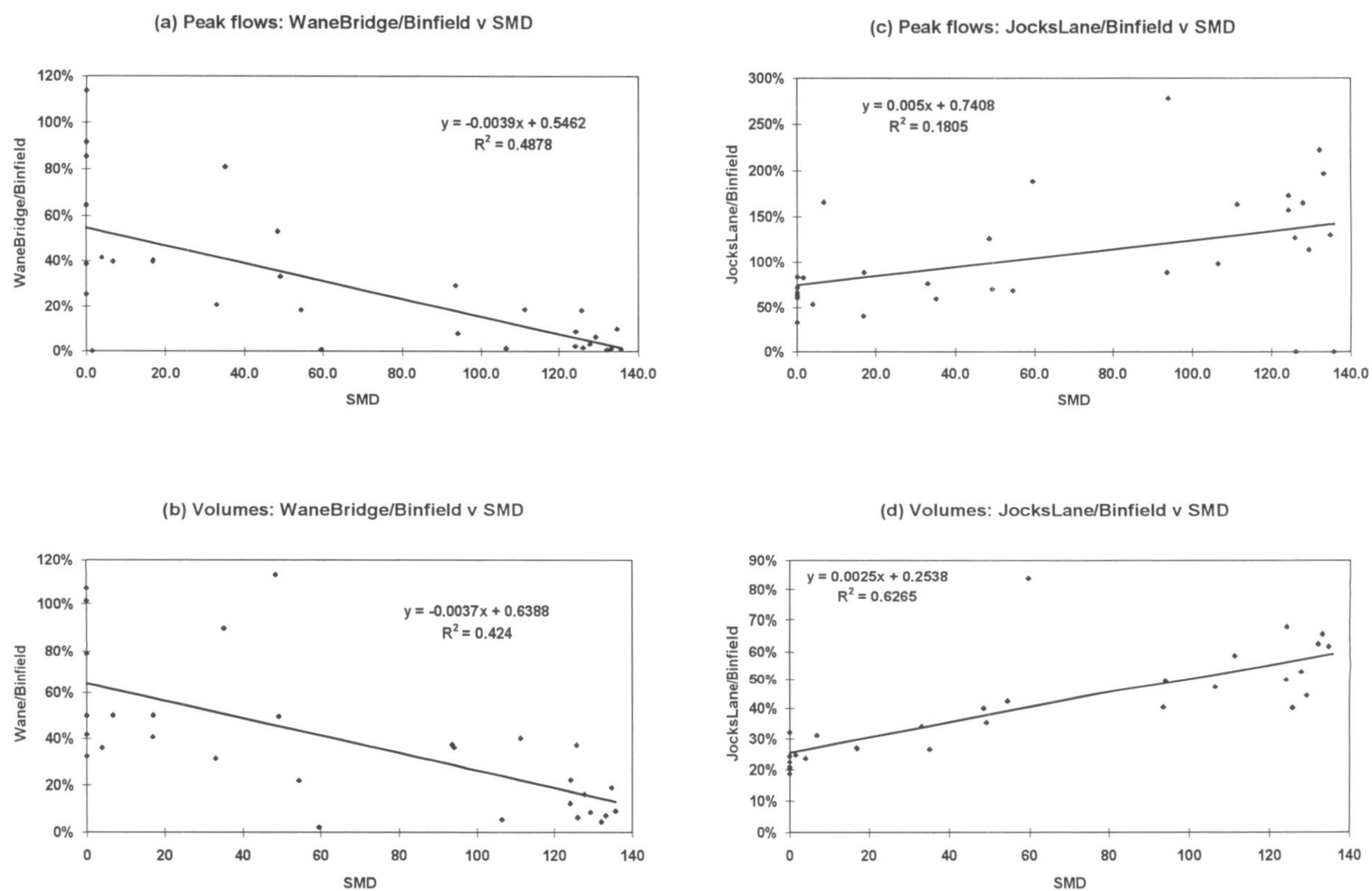

(a) Peak flows: WaneBridge/Binfield v SMD
y = -0.0039x + 0.5462
R^2 = 0.4878
WaneBridge/Binfield
SMD
(c) Peak flows: JocksLane/Binfield v SMD
y = 0.005x + 0.7408
R^2 = 0.1805
JocksLane/Binfield
SMD
(b) Volumes: WaneBridge/Binfield v SMD
y = -0.0037x + 0.6388
R^2 = 0.424
Wane/Binfield
SMD
(d) Volumes: JocksLane/Binfield v SMD
y = 0.0025x + 0.2538
R^2 = 0.6265
JocksLane/Binfield
SMD

Figure 3.12

4. Model overview

4.1 GENERAL DESCRIPTION

The model SCHEME used in this study is a 'semi-distributed' flood routing model in which storm rainfall is propagated through the various subcatchments, channels and reservoirs that comprise a complex catchment. Thus variations in basin form and response between different parts of a catchment may be modelled, spatial variation in rainfall and other inputs may be incorporated, and runoff hydrographs at several points within a catchment may be determined, providing a basin wide appreciation of runoff characteristics. The description given below is intended to explain the basic principles of the model and to explain the function of the various model parameters. Fuller details of the model are given in Appendix A.

SCHEME draws its model procedures from many sources, including the Flood Studies Report (NERC, 1975), the Wallingford Procedure (DoE/NWC, 1981), the Australian RORB model (Laurenson and Mein, 1988), and previously unpublished work at IH. It includes a number of different options for modelling the various processes and how they vary between subcatchments. It was developed primarily for assessing floods in partly urbanised catchments that might include a number of flood storage ponds. However it is also well suited to analysing the effect of varied topography, to modelling 'strings' of reservoirs, and to assessing areal rainfall impacts. It does not consider backwater effects in channel routing. It has been applied to catchments ranging in size from 0.34 to 370 km^2 with a data timestep ranging from 2 to 60 minutes. It is programmed in FORTRAN77 and currently runs on PC's in a DOS-BOX.

Runoff modelling begins with distributing storm rainfall to subcatchments, and estimating the direct runoff and baseflow components using a continuous updating of antecedent precipitation. Baseflow is routed through a linear reservoir, re-combined with the direct runoff, and routed to the subcatchment outlet using a unit hydrograph approach. Channel routing uses a linear or non-linear form of the convection-diffusion wave equation. Normal (on-line) reservoir routing uses the standard level-pool equations combined with flexible storage and outlet/control rules. Off-line reservoirs are modelled as two or three 'coupled' reservoirs with free, drowned, or one-way interflows determined from the overflows, flap valves, etc. as appropriate. Reservoir characteristics may be adjusted at 'run time' in order to reduce outflow peak to some pre-set target. Depending on the level of detailed knowledge of the catchment, ranging from map data through detailed channel and flood plain data to extensive flow data, the basic models can be tuned and calibrated to improve model prediction.

The model is hybrid in form, having been developed to analyse urban impacts on observed rainfall-runoff response in mixed catchments. Thus while some components are defined in parametric form, others such as baseflow, percentage runoff and response time may be assessed and adjusted through a more analytical approach. The distributed routing procedures are based on the recommendations of the Flood Studies Report (NERC, 1975) and Flood Studies Supplementary Report 16 (IH, 1985) with parameters defined from subcatchment characteristics. These parameters can be adjusted individually for each subcatchment/channel, but are usually adjusted over all subcatchments based on a number of global model parameters (factors). These comprise four routing parameters, two parameters to adjust for the effects of urbanisation, one overall rainfall loss parameter plus one for each flow gauge, and two baseflow parameters for each flow gauge. Normally the baseflow and all but one of the loss parameters are fixed by prior analysis of observed rainfall-runoff data. Two of the routing parameters are relatively insensitive, which leaves just two routing parameters, one loss parameter, and maybe the urbanisation parameters to be found by optimisation. An automatic optimisation routine is included, which may be used together with, or instead of, manual/visual methods.

The model operates in four modes:

- CHECK, in which catchment data and flow data are read to confirm their format and then runoff and baseflow volumes are analysed
- FIT, in which global parameters may be optimised to improve the fit to observed rainfall-runoff data
- TEST, in which the fit of a single set of global parameters may be tested against observed rainfall-runoff data, and
- DESIGN, in which the fitted model is used to predict response from a DESIGN rainfall event.

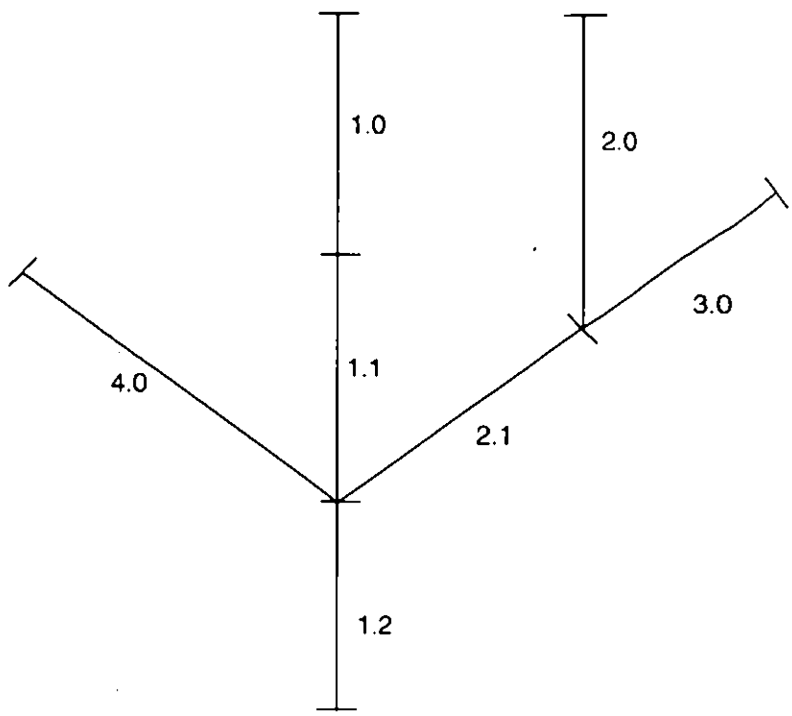
Design rainfall may be input directly, or specified as a combination of return-period, profile and duration according to the UK Flood Studies Report (NERC, 1975) rainfall model.

4.2 NODAL REPRESENTATION OF THE CATCHMENT

To apply the model, the catchment is divided into a number of subcatchments on the basis of the channel network and surface characteristics. The channel network is represented by a series of nodes placed at the subcatchment outlets, at significant confluences, reservoirs, gauging stations, and other points of interest. The *runoff calculation sequence* begins at the most upstream node on the main channel. The hydrograph is derived using the subcatchment model and routed to the next node downstream using the channel model. Response from the inter-nodal subcatchment is estimated and added to the current hydrograph. At confluences the current hydrograph is stored and the calculations begin again at the top of the incoming tributary branch. When modelling of that branch is complete, the stored hydrograph is added to the current hydrograph and the calculations proceed as before down the main channel. At gauging stations the observed and current hydrographs may be compared and various goodness-of-fit statistics calculated. The gauged hydrograph may optionally have already been used to define *runoff volumes* for subcatchment routing, and similarly it may now be used for parameter optimisation or even adopted as the current hydrograph. This *calculation sequence* is repeated down to the final node. Areas below the final gauging station would use *runoff volume* information from the final gauging station. If no flow hydrographs are specified for defining *runoff volumes*, the user is requested to supply appropriate 'design values' for baseflow and percentage runoff parameters.

The *runoff calculation sequence* described above is input to the model as an ordered list of '*branch.reach*' codes for the various nodes. These codes (familiar to sewerage engineers) serve as labels for the node, the upstream inter-nodal subcatchment and the channel. Within SCHEME they are also used to reference gauging station nodes and nodes for hydrograph output. Figure 4.1 gives a nodal representation of a simple example catchment, together with the *calculation sequence* of *branch.reach* codes. The codes start with the mainstream (normally *branch* 1) and work from upstream to downstream (normally incrementing *reach* from an initial value 0). At confluences the current *branch* is suspended while the tributary nodes are traversed (also upstream to downstream) using a new (incremented) *branch* and (re-initialised) *reach*. This procedure is nested as necessary for any sub-branches. There is no limit on the number of branches that can meet at a confluence, but the maximum number of confluences that may be nested within a branch is currently fixed at four. At the next node below a confluence, the *branch* code returns to that of the first (mainstream) *branch* to enter the confluence, and the *reach* code is incremented with respect to that *branch*. A node is not required at the top of the outgoing reach from a confluence unless the combined

Nodal representation of sample catchment



Calculation sequence

Branch reach	MODEL OPERATION
1.0	SUBCAT 1.0
1.1	CHAN 1.1, ADD SUBCAT 1.1
2.0	STORE1.1, SUBCAT 2.0
3.0	STORE 2.0, SUBCAT 3.0
2.1	ADD STORED 2.0, CHAN2.1, ADD SUBCAT 2.1
4.0	STORE 2.1, SUBCAT 4.0
1.2	ADD STORED 2.1, ADD STORED 1.1, CHAN 1.2, ADD SUBCAT 1.2

Where
SUBCAT = Model subcatchment hydrograph
CHAN = Route hydrograph along channel

Figure 4.1

hydrograph at the confluence is specifically required, in which case a special junction node is provided.

In addition to a *branch.reach* code, each node has up to three 'flags' to indicate special features. The first flag indicates reservoir or junction nodes (involving no subcatchment or channel routing). The second and third flags invoke specific subcatchment and channel routing options (see Appendix A).

The example *branch* and *reach* codes shown on Fig. 4.1 are (for simplicity) single digits, but each may extend to 3 digits. Current program limits allow a maximum of 100 nodes, 80 subcatchments and 4 nested confluences. In practice, having *branch* and *reach* codes increase sequentially is convenient for the user, but not necessary to the program. *Branch* codes can take any value not already active (or stored), while *reach* codes need only be distinct within the *branch*. This allows the subcatchment configuration to be changed without the need to redefine all the *branch.reach* codes.

4.3 MODELLING PROCEDURES AND PARAMETERS

The first stage of the SCHEME model involves the assessment of surface (direct) runoff and baseflow volumes. Baseflow is generally a small component of an overall flood hydrograph, but early (unpublished) studies with SCHEME showed that simple 'baseflow separation' procedures such as used in the FSR could also remove what was originally direct runoff from remoter parts of the catchment. SCHEME thus models both baseflow and surface runoff, adopting a 'contributing area' approach with drainage to baseflow and surface runoff occurring from the same 'active area'. Baseflow is modelled using a linear reservoir lag (K) with recharge (R) proportional to direct runoff (i.e. for every x% of rainfall that goes to surface runoff, another Rx% goes to baseflow recharge). Outflow from the baseflow reservoir is added to the direct runoff and routed through the subcatchment and channel network. Provided that the routing models are linear, and that the recharge and reservoir lag do not vary between subcatchments, the baseflow parameters can be defined directly from an observed hydrograph (without specifying the exact form of the direct runoff model) and applied at a subcatchment level. Thus the baseflow parameters can be determined by local analysis rather than global optimisation. If linearity and spatial homogeneity are not maintained, the parameters determined from the downstream hydrograph cannot be applied at the subcatchment level and still reproduce exactly the same downstream baseflow response, but the departures have so far proved to be small. While there are some concerns with the approach, it does provide a reasonable method for assessing direct storm runoff volumes. The user usually derives the baseflow parameters while running the model in CHECK mode, and has the opportunity to adjust them in any later model runs.

Direct runoff at each timestep through a storm is modelled using a constant coefficient applied to rainfall plus a varying coefficient dependent on the Antecedent Precipitation Index, API. As in the FSR, API is an exponential decay applied to antecedent rainfall, and updated through the storm based on an equivalent daily decay rate of 0.5. This rate is currently fixed, but could easily be changed in future versions to adopt aspects of the HydroWorks 'new runoff model' where the decay rate depends on soil type. The SCHEME model parameters are: IARC, defining a constant runoff coefficient from impervious areas; and CRCF, defining the proportion of pervious area to be modelled as a constant runoff coefficient. In FIT and TEST modes, overall percentage runoff is usually 'forced' to give the observed runoff volume at designated gauging stations (taking into account rainfall, API and impervious area variations between subcatchments). Note that in this case TEST mode simply checks the routing parameters, but for a full TEST the user may specify the baseflow and percentage runoff parameters directly. In DESIGN runs, where no downstream flow data are available, baseflow and percentage runoff parameters <u>must</u> be specified directly.

Subcatchment runoff routing is by unit hydrograph. Several standard unit hydrograph models are included (FSR, CIRIA, Nash), but in most cases the FSSR16 model would be used, with unit hydrograph shape defined by QpTp=220, and time to peak related to stream length, and slope, climate (the average annual rainfall, SAAR) and the URBAN proportion. The SCHEME model parameters UHSF and UHTF allow global adjustment of unit hydrograph shape and time to peak across all subcatchments, while the parameter UHUA allows global adjustment for urban effects (as $Tp_u/Tp_r = (1+URBAN)^{**}(-2^*UHUA)$)

Channel routing uses a convection-diffusion model, based on either a fixed or flow related channel delay and attenuation. Normally, unless there is particular concern for flow related impacts (e.g. overbank flooding) the fixed delay form would be used. Wave celerity is defined from weighted time to peak or lag estimates at the upstream and downstream end of the channel, and attenuation is defined from mean flow peak and channel slope and breadth. Model parameters CHCF and CHAF allow global adjustment of the wave celerity and attenuation estimates in each subcatchment.

Reservoirs may be on-line (where all flow passes through the reservoir, and downstream discharge depends directly on reservoir storage), or off-line (where excess flow above some limit is diverted to storage until the inflow falls and the reservoir can drain back to the channel). On-line reservoirs are modelled by the normal 'level pool' equations combining the storage-head and outflow-head relationships. Off-line reservoirs are modelled as two or three 'coupled' reservoirs, involving (a) flow diversion from an inlet tank (e.g. a throttle pipe and side-weir), and return flow via an outlet tank (e.g. with flap valves), or (b) both diversion and return from a single combined tank. In either case, free/drowned/one-way interflows are determined as appropriate. For both on-line and off-line ponds, storage-head relationships may be defined by (i) a power-law equation or (ii) a table of data points, and outflow-head relationship may be defined by (i) a number of power-law equations (for different controls and ranges of control), or (ii) a table of data points. In equation form, reservoir characteristics may be left undefined - to be adjusted at 'run time' in order to reduce outflow peak to some pre-set target.

All the subcatchment, channel and reservoir data needed to define the flow routing through the catchment (e.g. areas, soils, URBAN fractions, channel lengths, slopes, reservoir areas, controls) are stored against the ordered *branch.reach* codes in a catchment data file. Rainfall and flow data are stored in storm data files. The model accesses selections from these files, requests certain on-line information (e.g. the nodes at which hydrograph output is required; the nodes at which hydrographs are available for fitting; the parameters to optimise and their respective ranges), and plots model hydrographs. The hydrographs may be saved to file or other previously saved hydrographs may be read for comparative plotting. At the end of a model run, the program offers a choice to change run type, parameters, storm, or catchment.

Apart from the flexibility in subdividing the catchment, in the baseflow modelling, in selecting subcatchment and channel routing options, and in selecting gauges to use in determining overall runoff volumes, SCHEME has seven main parameters. These are:

CRCF	the proportion of pervious area runoff taken as a constant proportion of rainfall (the remainder varies with API)
UHTF, UHSF	global factors for unit hydrograph time to peak and shape
CHCF, CHAF	global factors for channel celerity and attenuation
UHUA	urban adjustment for unit hydrograph response time, and
IARC	runoff coefficient for impervious areas.

The model includes a flexible parameter optimisation procedure which allows combinations of parameters to be fixed, varied in steps, or optimised against one of five objective measures of model fit.

Note that volume 'forcing' is normally used to define the pervious area runoff. However, if the observed runoff volume is less than would be given by the product of IARC and the rainfall over the impervious area, then pervious area runoff is set to nil, CRCF becomes redundant, and IARC is reduced accordingly. Conversely, in the extremely unlikely case that the derived pervious area runoff coefficient exceeds IARC, then IARC is increased accordingly.

5. Observed and modelled response of catchment

5.1 GENERAL STRATEGY

The main objectives of this project were to develop a flood estimation method for mixed urban/rural catchments, and verify the method through a case study of the Bracknell catchment. To these ends, the linked subcatchment model SCHEME has been developed (as outlined above in chapter 4, and described in detail in Appendix A), and an extensive four year programme of data collection followed in the Cut catchment (see Chapter 3 and Appendix B). Chapter 3 has presented some basic comparisons of flood response between different subcatchments of the Cut, based on 31 selected storm events. For modelling purposes these 31 events have been separated into 16 fit events and 15 test events. Since no significant change in catchment response was expected over the 4-year monitoring period, the events were split alternately, with the odd numbered events used for fitting and the even events used for testing. Although no assessment was made of the events prior to the split, the separation in terms of size and season of occurrence has proved remarkably balanced (see Table 5.1). No modelling of any sort was performed on the test events until the very end of the study, when the 'best fit' parameters obtained for the fit events were applied to the test events.

Table 5.1 Fit and Test events: number of events in specified categories

Set	Spring	Summer	Autumn	Winter	No of top 5 floods in set				No of top 10 floods in set			
	Mar-May	Jun-Aug	Sep-Nov	Dec-Feb	Wane	Warf.	Jocks	Binfld	Wane	Warf.	Jocks	Binfld
Fit	2	3	6	5	3	2	3	2	4	6	6	5
Test	1	4	5	5	2	3	2	3	6	4	4	5

Given the general problems of data accuracy, the majority of the modelling effort has concentrated on the accuracy of flow estimation at the main EA gauge at Binfield, though some results from comparisons at other gauges are presented. The model studies may be classified as:

- A 'base-line' study, using a fairly conventional application of the FSR unit hydrograph model, treating the whole Binfield catchment as a single lumped unit.
- A detailed SCHEME model, treating the catchment as 42 sub-catchments, with 18 flood storage ponds. The hypothesis was that this modelling would give an improved fit at Binfield, and also estimate flows at other locations within the catchment.
- A split record test for each model, with relationships for the volumetric and routing parameters determined from the 'fit events' applied in modelling the 'test events'.
- A demonstration, using SCHEME, to assess drainage strategy. Based on the 'fit events' and the 'best fit' parameters, the effect of each pond on peak flows has been quantified, and the effect of all ponds together on peak flows has been assessed.

Although described as the FSR model, the 'base-line' study was in fact based on SCHEME, but using a single subcatchment. Thus

- the SCHEME algorithm was used to determine catchment average storm rainfall from the individual gauged depths and profiles,

- the SCHEME baseflow model was used, and
- the 'best fit' unit hydrograph was derived by optimisation not matrix inversion.

These departures from the FSR model are believed to have minimal impact, and they in no way invalidate the usage as a 'base-line' lumped model.

Within SCHEME, optimisation must currently be applied to each storm individually, and differences in parameter values between storms either averaged or explained by reference to some other factor (such as SMD). An iterative optimisation technique is used, climbing the 'hill' formed by 'goodness-of-fit' at the various trial parameter values. When several parameters are optimised together, interaction and trade-offs are usually obtained between the different parameter values. The conventional optimisation strategy is thus successively to fix or determine relationships for the least variable parameters until plausible relationships are found for the remaining parameters.

The optimisation method used by SCHEME requires that each parameter is given an upper and lower bound, which not only retain the parameter within reasonable values, but define the allowable convergence error. Applying these bounds helps limit the effect of trade-offs between extreme parameter values, and as storms are optimised individually, it is usually apparent if a parameter has been unduly restricted. In that case, the bounds might be relaxed, or re-assessed through a series of sensitivity tests.

Optimisation in SCHEME can be based on one of several 'goodness-of-fit' criteria, including simple and weighted forms of the root-mean-square (RMS) error between the observed and modelled hydrographs. This study has used the simple RMS error, expressed as a percentage of observed peak flow. A criterion more directly targetted at high flows might have been more appropriate, but RMS error does tend to fix peaks, since larger errors are usually associated with high flows. Where volume or baseflow errors exist, enlarged RMS errors can occur, so optimisation has always included 'volume forcing' (proportional runoff and baseflow determined from the observed data). However, because RMS values have also been quoted at gauges used only for comparison (i.e. without volume forcing), the correlation coefficient (R^2) which compares only shape and not scale has also been quoted.

The catchment data used in the modelling are described in Appendix B; a schematic representation of the catchment is given here as Fig. 5.1, showing the *branch.reach* codes used by SCHEME to represent nodes and subcatchments.

5.2 BASEFLOW IDENTIFICATION

The first stage in the modelling is the identification of baseflow, and hence of surface runoff volumes from which to define runoff coefficients. In CHECK mode, SCHEME provides an interactive procedure for identifying an appropriate 'linear reservoir' delay for the event recession, and optimising the 'recharge factor' necessary to move from the baseflow prior to the event to meet asymptotically with the event recession. (Prior to this study, the user needed to define the exact point on the recession that marked the 'return to baseflow'.) The same baseflow 'separation' was used with the lumped 'FSR' approach and the more detailed SCHEME subcatchment approach.

At the start of the modelling study it was intended to fit the hydrographs at Wane Bridge, Warfield House, Jocks Lane, and Binfield. Thus baseflow delay and recharge were sought for each event at each gauge. Parameters derived for individual storms showed considerable

Schematic representation of catchment

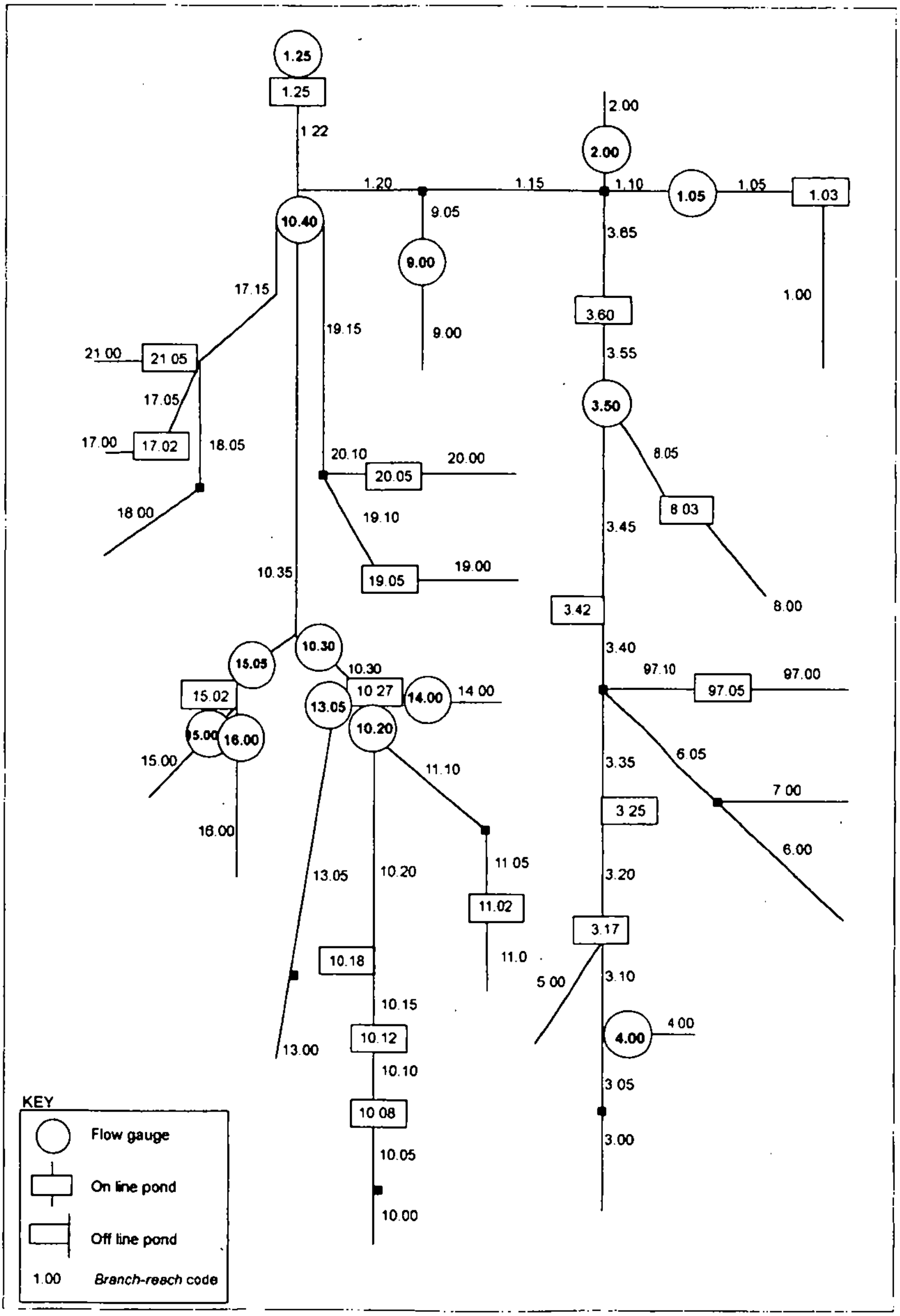


Figure 5.1

variability and also some cross correlation. Based on the early runs, a standard delay of 60 hours was therefore selected and applied to each gauge. Different delays might have been appropriate for each gauge, but this would have introduced difficulties in defining the delay to be used for the areas between the gauges.

With the fixed 60 hour delay, the recharge factor was determined for each gauge and fit event. Figure 5.2 shows the separation for four sample events at Binfield, where (a) is event 5, the largest fit event, (b) is event 11, a small event, (c) is event 17, the second largest, and (d) is event 25, a sequence of modest events. The separations appear quite believable. Note that rainfall data play no part in estimating baseflow, though the user could specify a target area for 'return to baseflow' based on some typical 'time from end of rainfall' as used in the FSR. With the baseflow identified, SCHEME evaluates percentage runoff (based on average rainfall on the upstream subcatchments). The derived values of recharge factor (R) and proportional runoff (PR) for each gauge and event are given in Table 5.2, along with initial flow rate at the start of the event (Q0, m^3/s).

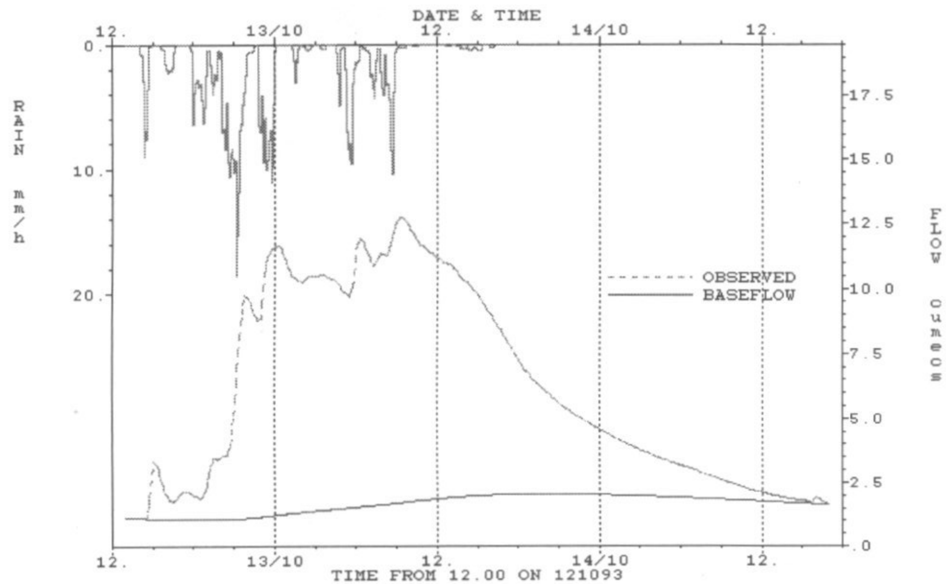
Table 5.2 Baseflow and percentage runoff parameters

		Wane Bridge 18.5 km²			Warfield House 12.3 km²			Jocks Lane 12.6 km²			Binfield 51.9 km²		
no	**SMD**	**R**	**Q0**	**PR**	**R**	**Q0**	**PR**	**R**	**Q0**	**PR**	**R**	**Q0**	**PR**
1	127.8	(1.)	0.018	0.02	0.40	0.184	0.27	0.33	0.054	0.17	0.71	0.118	0.06
3	124.3	0.00	0.146	0.00	1.00	0.298	0.47	0.00	0.133	0.24	1.40	0.271	0.07
5	35.1	0.29	1.369	0.80	0.50	0.388	0.68	0.27	0.210	0.43	0.50	1.133	0.33
7	0.0	0.52	2.121	0.82		-		0.75	0.302	0.41	1.21	2.186	0.26
9	0.0	0.00	0.000	0.70		-		0.89	0.160	0.32	1.22	0.698	0.27
11	6.9	1.04	0.077	0.41	0.56	0.361	0.69	1.04	0.136	0.27	1.45	0.466	0.20
13	124.2	(2.)	0.030	0.01	0.00	0.262	0.29	0.00	0.106	0.18	0.00	0.323	0.06
15	94.1	(2.)	0.128	0.08	0.00	0.349	0.64	0.43	0.059	0.24	1.07	0.238	0.09
17	17.0	0.29	0.274	0.48	0.72	0.202	0.28	0.45	0.130	0.35	0.64	0.487	0.29
19	1.5				1.69	0.335	0.32	0.79	0.175	0.37	1.00	0.788	0.33
21	132.1	(50.)	0.024	0.00	0.00	0.125	0.17	0.44	0.053	0.58	0.75	0.145	0.11
23	126.1	2.81	0.026	0.01	0.90	0.068	0.14				0.61	0.135	0.09
25	33.0	0.62	0.230	0.17	1.00	0.258	0.26	0.54	0.210	0.28	0.89	0.703	0.17
27	4.0	0.51	0.096	0.34	1.21	0.085	0.27	0.71	0.116	0.27	0.94	0.392	0.27
29	129.3	0.83	0.010	0.02	0.17	0.100	0.21	0.26	0.049	0.17	0.46	0.107	0.09
31	93.6	1.03	0.159	0.11	0.00	0.643	0.62	0.30	0.061	0.27	0.57	0.270	0.13

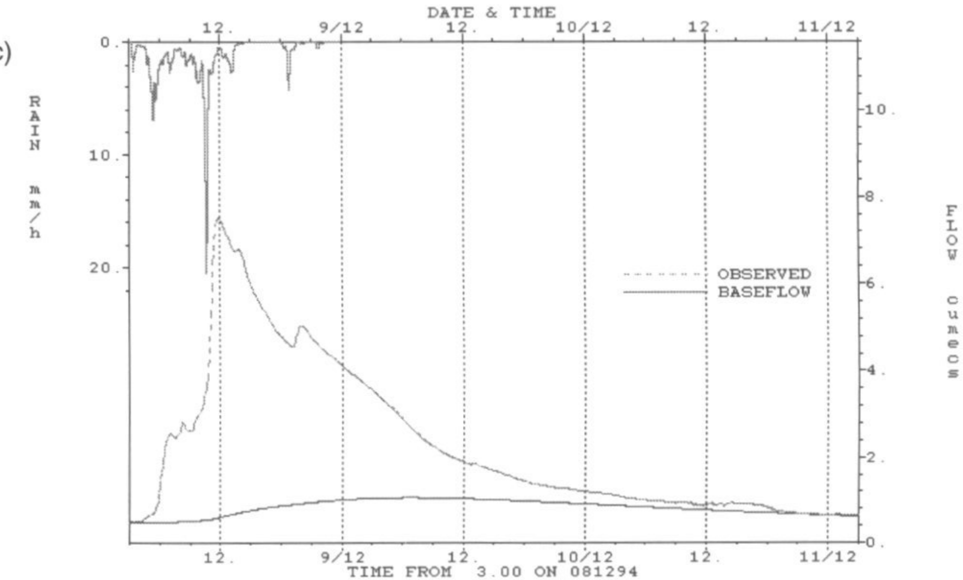
Note: The slow response at Wane Bridge meant that a long recession period was required to return to baseflow. On a few occasions recharge was estimated based on an assumed recession extension. These results are shown in brackets and are provided for comparison only.

Baseflow identification: sample plots (5,11,17 and 25)

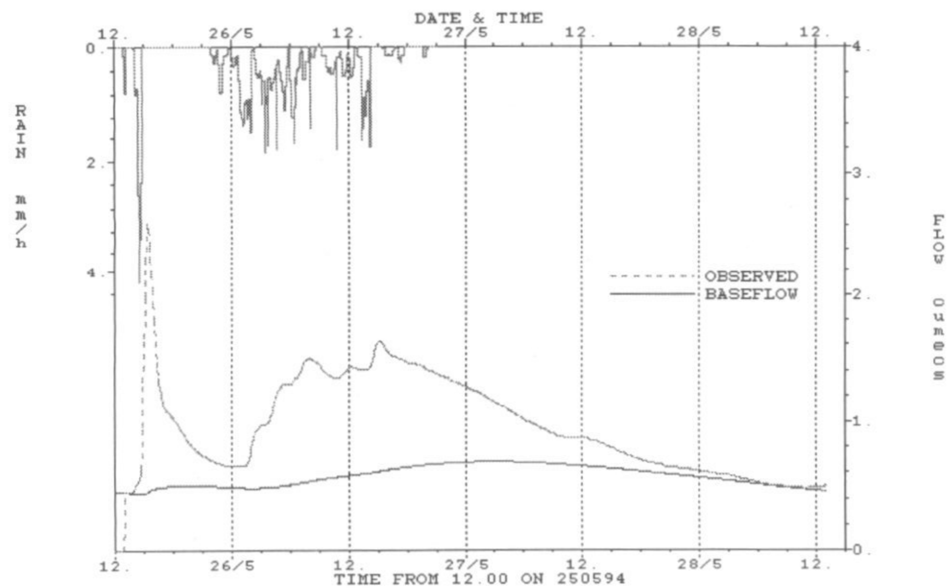
a)



c)



b)



d)

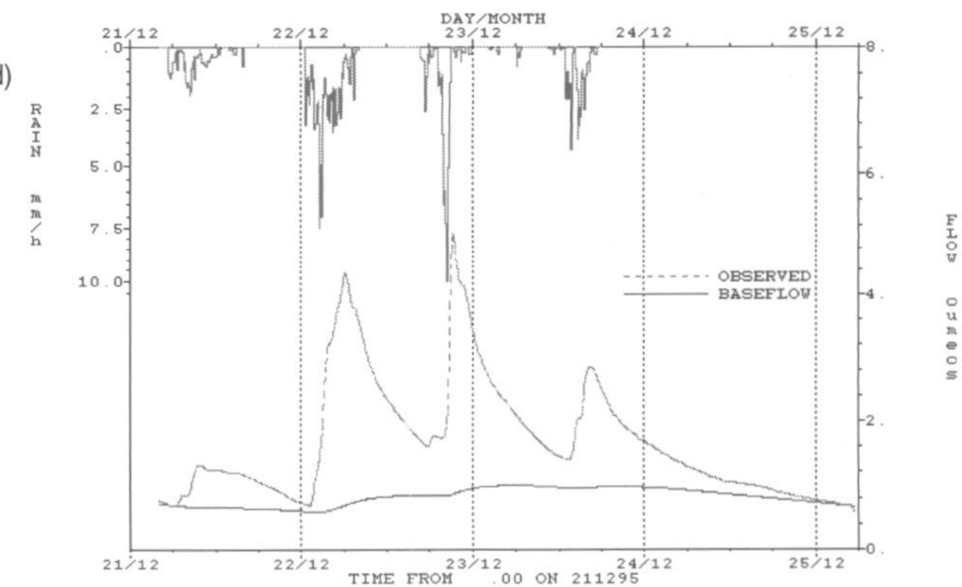


Figure 5.2

For reasons discussed below, only the Binfield results have been used in the modelling studies, and Fig. 5.3 shows how the event initial flow, Q0, recharge factor, R, and proportional runoff, PR, all reduce with SMD (as the catchment dries). Note that actual recharge is defined by the product of R, PR and rainfall depth. Plots for the other gauges are given in Fig. 5.4, where it may be noted that initial flows at Warfield House seem to be approximately double those at Jocks Lane, while in dry conditions, initial flows at Wane Bridge are effectively zero. Note also that proportional runoff at Wane Bridge is very variable, but at Jocks Lane is fairly constant. However, more specific conclusions should be treated with some caution.

As largely expected, given the preliminary data screening results of Table 3.4, initial modelling based on these parameters showed, for almost every storm, greater direct runoff from Wane Bridge, Warfield House and Jocks Lane combined than was measured at Binfield. Negative runoff from the intervening 8.5 km^2 of area is not physically possible, and is not allowed in the model (it is set to zero). For this reason, it was decided that for the rest of this study all model fitting would be based on just the Binfield data, with the other gauges used only for comparisons. It should be remembered that there have been a number of problems in developing ratings for Wane Bridge and Warfield House (see Appendix B), and these results suggest some further modifications are required. In this respect, improved use might be made of the few spot gaugings available.

The fitted parameter values for each event at Binfield (see Table 5.2), were therefore applied throughout the catchment. Currently there is no allowance in SCHEME for urbanisation impacts on baseflow, so lower recharge factors for urban areas (the Jocks Lane figure is typically 60% of that at Binfield) have not been applied. The best-fit equations, taken from Fig. 5.3 and given below, have been used to estimate parameters for the test events - except for initial flow, where using the observed value at the start of the event was considered acceptable.

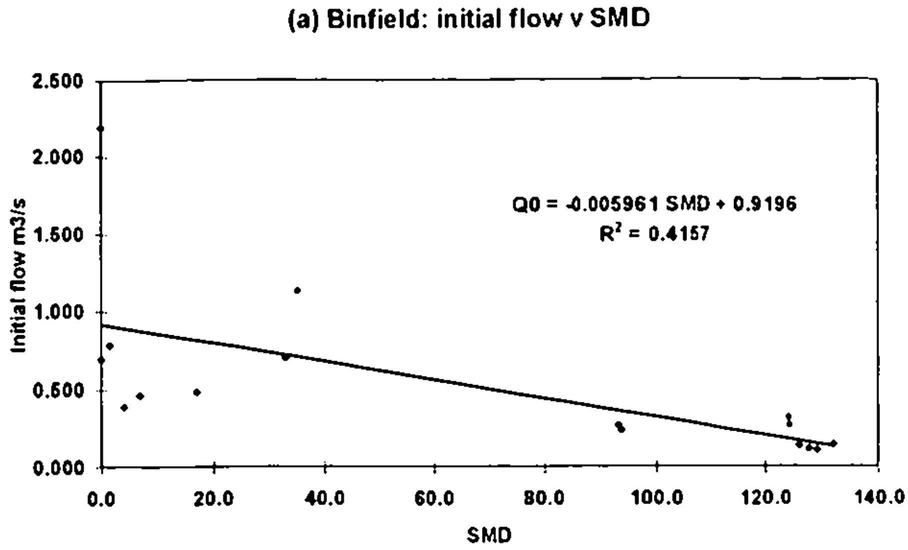
$$R = 1.0437 - 0.00313\ SMD \qquad R^2=0.215$$
$$PR = 0.2801 - 0.00161\ SMD \qquad R^2=0.807$$
$$Q0 = 0.9196 - 0.00596\ SMD \qquad R^2=0.416$$

These equations should not be interpreted as having general applicability to other catchments, being based on just 16 events at Binfield. The R and PR equations do however represent the equivalent of using local data to define Standard Percentage Runoff (SPR) in the FSR method (see FSSR13). It may be noted that the constant 0.28 is rather less than the value of 0.40 recommended in the FSR for soil type 3, but this may in part reflect the additional runoff volume due to the increase in baseflow during a storm that SCHEME allows but the FSR model does not.

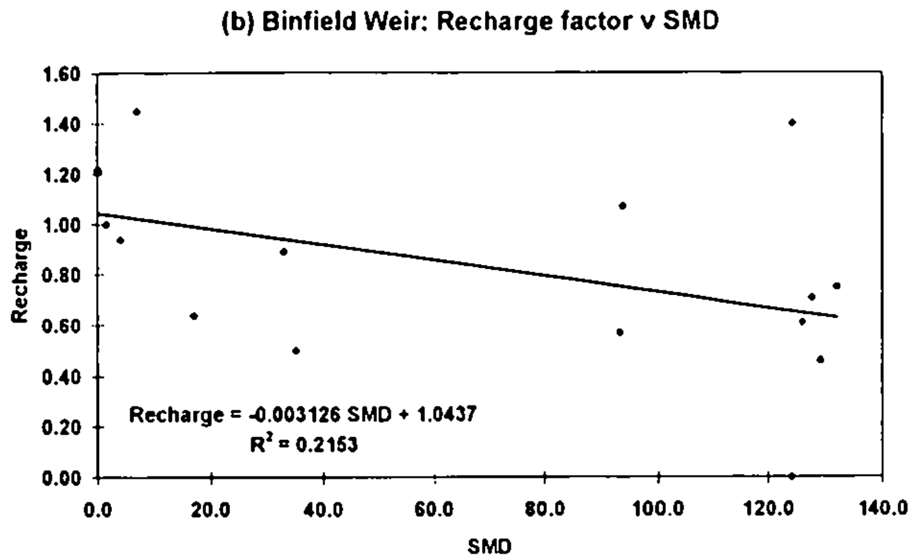
5.3 THE LUMPED 'FSR' MODEL

As discussed above, the FSR lumped unit hydrograph model has been applied as a single subcatchment implementation of SCHEME. Note that the baseflow parameters were exactly as described for Binfield in Table 5.2 above, but because rainfall determined for the catchment as a whole differed slightly from the weighted average of the subcatchments, the PR values obtained were slightly different. A fuller description of how SCHEME derives subcatchment rainfall is given in Appendix A, but on average, the lumped catchment rainfall was 1.01 times that derived from the 42 subcatchments.

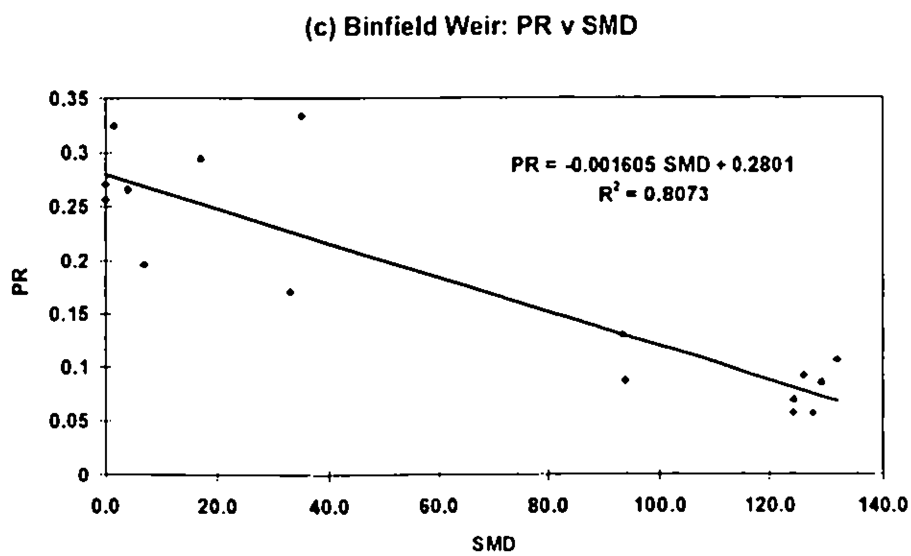
Of the seven SCHEME parameters, only four are of relevance in a single subcatchment implementation:



(a) Binfield: initial flow v SMD
Initial flow m3/s
Q0 = -0.005961 SMD + 0.9196
R² = 0.4157
SMD



(b) Binfield Weir: Recharge factor v SMD
Recharge
Recharge = -0.003126 SMD + 1.0437
R² = 0.2153
SMD



(c) Binfield Weir: PR v SMD
PR
PR = -0.001605 SMD + 0.2801
R² = 0.8073
SMD

Figure 5.3

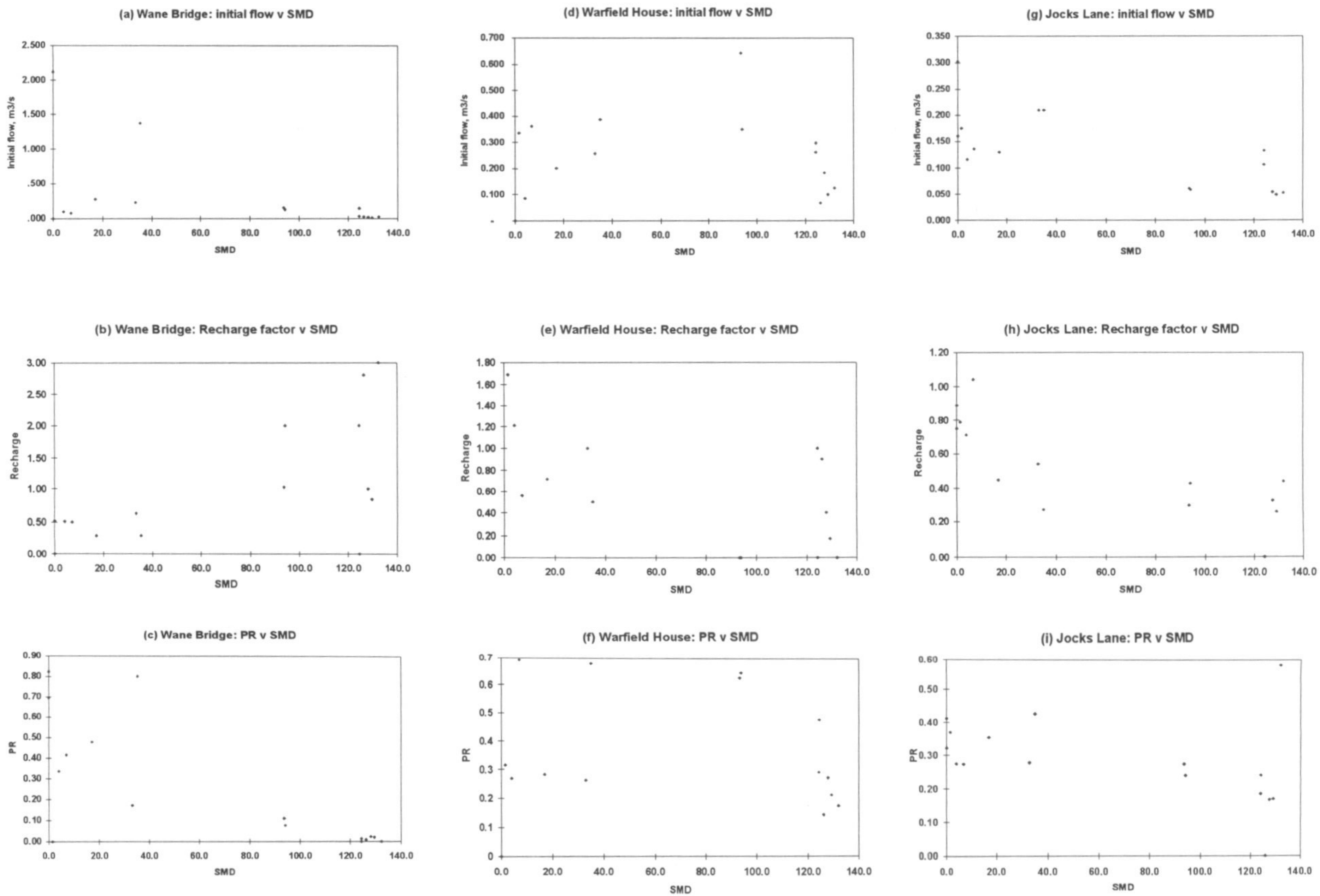

(a) Wane Bridge: initial flow v SMD
Initial flow, m3/s
SMD
(d) Warfield House: initial flow v SMD
Initial flow, m3/s
SMD
(g) Jocks Lane: initial flow v SMD
Initial flow, m3/s
SMD
(b) Wane Bridge: Recharge factor v SMD
Recharge
SMD
(e) Warfield House: Recharge factor v SMD
Recharge
SMD
(h) Jocks Lane: Recharge factor v SMD
Recharge
SMD
(c) Wane Bridge: PR v SMD
PR
SMD
(f) Warfield House: PR v SMD
PR
SMD
(i) Jocks Lane: PR v SMD
PR
SMD

Figure 5.4

UHTF,UHSF	the unit hydrograph time to peak and shape factors,
CRCF	the proportion of runoff taken at a constant percentage of rainfall, (the remainder varies with Antecedent Precipitation Index, API)
and IARC	the impervious area runoff coefficient,

The unit hydrograph urban adjustment parameter UHUA has been set equal to one (for a single subcatchment implementation it is obviously constant throughout the catchment, and thus simply adjusts the effective UHTF value). IARC, the impervious area runoff coefficient, has been set to 0.7 (i.e. the FSSR5/FSSR16 recommended value). In a single subcatchment implementation, IARC overlaps with CRCF, creating an area of (high) constant percentage runoff. An equally good fit to a specific storm can normally be obtained for a range of IARC values by adjusting CRCF. Note that these urban parameters are intended to model differences in runoff behaviour between catchments or subcatchments, and cannot be optimised in one-off, lumped applications.

Fitting the lumped model was thus reduced to determining optimum values for UHTF, UHSF and CRCF such that the RMS error between modelled and observed flows was minimised. During optimisation, minimum permissible values of 0.1 and 0.2 respectively were specified for UHTF and UHSF, while CRCF was optimised between 0.01 and 1.0 - i.e. proportional runoff varied between, effectively, direct dependence on API, and invariance during the event. The CRCF limits are physically based, but those on UHSF and UHTF were imposed to resist too extreme a departure from the FSR expected values of 1.0. The first column group of Table 5.3 shows the parameter values obtained through optimisation and the corresponding RMS error and correlation coefficient R^2.

Table 5.3 Fit details for 'FSR' model

Ev no	1. Optimising 3 parameters IARC=0.7					2. Optimising 2 parameters UHSF=0.2				3. Optimising UHTF CRCF=1.0		
	UHTF	UHSF	CRCF	RMS	R^2	UHTF	CRCF	RMS	R^2	UHTF	RMS	R^2
1	0.127	0.20	1.00	5.8	0.912	...	...	5.8	0.912		5.8	0.912
3	0.384	1.20	-	12.1	0.803	0.100	-	13.9	0.744		13.9	0.744
5	0.405	0.20	1.00	5.5	0.984	...	...	5.5	0.984		5.5	0.984
7	0.425	0.20	1.00	7.1	0.930	...	...	7.1	0.930		7.1	0.930
9	0.307	0.20	1.00	4.2	0.987	...	...	4.2	0.987		4.2	0.987
11	0.439	0.20	1.00	9.4	0.813	...	...	9.4	0.813		9.4	0.813
13	0.118	0.28	-	5.2	0.940	0.100	-	5.4	0.935		5.4	0.935
15	0.173	0.59	0.01	7.5	0.936	0.100	1.00	8.1	0.882		8.1	0.882
17	0.361	0.20	1.00	6.6	0.970	...	...	6.6	0.970		6.6	0.970
19	0.312	0.20	1.00	6.9	0.964	...	...	6.9	0.964		6.9	0.964
21	0.125	0.35	0.01	10.4	0.836	0.100	0.01	10.7	0.797	0.100	10.7	0.796
23	0.144	0.20	0.01	9.1	0.870	...	...	9.1	0.870	0.157	9.2	0.867
25	0.199	0.20	0.01	6.6	0.943	...	...	6.6	0.943	0.206	7.0	0.932
27	0.320	0.20	0.29	8.1	0.952	...	...	8.1	0.952	0.447	9.9	0.904
29	0.130	0.21	1.00	9.4	0.831	0.126	1.00	9.4	0.830		9.4	0.830
31	0.224	0.20	1.00	9.1	0.938	...	...	9.1	0.938		9.1	0.938
av				7.7	0.913			7.9	0.903		8.0	0.900

As described above, the optimisation strategy now involved fixing one parameter and re-optimising the other two. Note however that 11 of the 16 events had reached the limiting UHSF value (0.2), while for CRCF, 9 had reached the upper limit of 1.0 (i.e. constant proportional runoff), 3 had reached the lower limit (just 1% at a constant proportional runoff), and another 2 could not define a value (the observed runoff was less than the product of IARC and the impervious area rainfall). Of these bounded values, UHSF was the most unexpected. Volume considerations for the triangular unit hydrograph requires QpTb=555 (where Qp is the peak and Tb is the base), so the FSR equation QpTp=220 (where Tp is the time to peak) defines time to peak as approximately 40% of the time base. A UHSF factor of 0.2 thus equates to a much more skewed unit hydrograph, with a time to peak of about 8% of the time base. This excessively skewed optimum shape may be needed in a mixed catchment to reflect the very fast response from the urban area combined with the more sluggish response from rural areas. It may also reflect the short timestep used in this study (5 minutes) in relation to the predicted time to peak (UHTF*5.67 hours).

Further sensitivity tests showed that reducing UHSF below 0.2 had a diminishing effect, and it was decided to fix on a value of 0.2. Just 5 events needed re-optimising, giving the second column group in Table 5.3. Note the generally reduced fit, with higher RMS and lower R^2 values, and note also that 4 of the events reached the lower limit on UHTF of 0.1. Reference back to Table 3.3 showed these events were all relatively small. Also, a more skewed unit hydrograph would need a shorter time to peak to preserve mean lag time.

Examination of the parameter values in the first and second column groups of Table 5.3 showed a wide range of both UHTF (from 0.1 to 0.44) and CHCF (from .01 to 1.0). However it was noted that the optimum UHTF seemed to be related to SMD (given in Table 5.2), with short times to peak corresponding to high SMD. An explanation might be that in dry conditions only the fast response from urban areas generally close to Binfield is observed, but in wet conditions the slower and more remote rural areas respond, slowing the average overall response time. In any case, the optimum UHTF values (for UHSF=0.2) were plotted against SMD as shown later in Fig. 5.6(a). The derived relationship is both understandable and adequately defined, and although a conventional FSR analysis would expect to determine a single Tp for the catchment, the derived variability has been accepted as a true effect of urbanisation. The relationship has thus been adopted as the overall 'best estimate' for UHTF, and only one parameter remained to be defined, CRCF.

From Table 5.3, with UHSF=0.2, just four events gave CRCF values that differed from unity, and no underlying trend in CRCF values could be discerned. The events were thus re-optimised with CRCF set to 1.0. Only event 27 showed any significant reduction in fit. However, the role of the CRCF parameter needs careful consideration. It is intended to answer the question "does proportional runoff increase as the catchment 'wets-up' during an event?" API is effectively the status of an upper soil moisture store that is replenished by rainfall, but drains by half during a period of 24 hours. CRCF defines how constant the pervious area runoff coefficient is, or conversely how much the runoff coefficient increases with soil moisture (API) during a storm. However, allowing the runoff coefficient to increase over a storm effectively transfers runoff from the start to the end of the storm, causing an apparent increase in lag time. Thus, particularly for single peaked events, a similar effect to a low CRCF value can be achieved by a higher UHTF value. Often it is only in complex sequences of peaks with clear differences in volume response between the successive peaks that reliable values of CRCF will be obtained. Inspection of all the fit events (see Fig. 5.5) showed events 25, 27 and 29 were clearly the most 'multi-peaked' events, and might be expected to require CRCF values less than one (observed hydrographs for events 3, 11, 13, 15, 21, and 23 meanwhile even suggest higher PR at the start of an event!).

Following from this discussion, it has been thought unwise to force CRCF values to 1.0. The parameters UHTF and CRCF from the first two column groups of Table 5.3 (with UHSF=0.2) have thus been adopted as the overall 'best-fit' for the FSR model, with the corresponding measures of fit, RMS and R^2, as given in the second column group. Averaging over all events, gave an RMS error of 7.9 and an R^2 of 0.903. Parallel results (not included here) showed that peaks flows were underestimated by an average of 20.5%. These values are compared with the results from the detailed SCHEME model in the next section, and the fit of both models to each event is shown in Fig. 5.5.

For the split record tests, UHTF has been estimated by the equation on Fig. 5.6a:

$$\text{UHTF} = 0.368 - .002\ \text{SMD} \qquad R^2=0.792$$

but as no reliable relationship between CRCF and storm depth, duration or SMD has been found, a fixed CRCF value of 1.0 has been adopted on the understanding that a lower value may be appropriate for multi-peaked events.

5.4 THE DETAILED SCHEME MODEL

5.4.1 Fitting the model

Optimisation of the detailed 42 subcatchment application of SCHEME followed the same procedure described for the lumped application. When there are several flow gauges within a catchment, SCHEME can allow optimisation to be based on any or all of them. However, as mentioned above, volume discrepancies between the gauges, combined with inconsistency in the data record at each gauge, led to the decision in this study to optimise model performance at Binfield alone. This has the advantage that the overall 'goodness-of-fit' at Binfield can be compared directly with that obtained from the lumped model. The fit obtained between modelled and observed response at the other gauges is an 'added benefit', and is discussed in a later section.

In order to restrict the number of SCHEME parameters needing to be optimised, unit values (i.e. expected values) have been applied to the unit hydrograph shape factor UHSF and the Channel Attenuation Factor CHAF. For the impervious area runoff coefficient IARC, values of 0.5, 0.6 and 0.7 have been applied, spanning the likely optimum value and allowing model sensitivity to be assessed. These tests showed that the sensitivity was low, and as the FSSR5/16 recommended value of IARC=0.7 generally gave the better fit to Binfield data, only those results are presented here. Of the four remaining parameters, limiting values of 0.3 and 3 were applied to UHUA, UHTF and CHCF, allowing a threefold deviation from the expected value of unity, and for CRCF limits of 0.01 and 1 were applied, as for the 'FSR' model. Table 5.4 gives details of the parameter optimisation for each event. Further discussion on the values obtained follows, but note:

- the general improvement in fit between Tables 5.3 and 5.4,
- for the second stage of optimisation CHCF was set equal to 3.0, and
- for the third stage of optimisation UHUA was set to the average value at stage 2 (i.e. 1.8).

Table 5.4 Fit data for detailed SCHEME model

Ev no	1. Optimising 4 parameters IARC=0.7						2. Optimising 3 parameters CHCF=3					3. Optimising 2 parameters UHUA=1.8			
	UHUA	UHTF	CHCF	CRCF	RMS	R^2	UHUA	UHTF	CRCF	RMS	R^2	UHTF	CRCF	RMS	R^2
1	1.529	0.598	3.000		3.5	0.967	1.537	0.599		3.5	0.967	0.836		3.5	0.966
3	2.195	0.300	1.398	1.000	11.7	0.885	0.950	0.300	0.623	11.3	0.908	0.786	0.600	12.1	0.905
5	1.886	1.410	0.640	1.000	4.0	0.991	1.291	1.609	0.773	4.9	0.987	1.673	0.621	5.6	0.983
7	2.245	1.596	0.977	1.000	5.2	0.959	1.857	1.654	0.011	5.6	0.953	1.618	0.010	5.6	0.953
9	1.455	1.245	2.958	0.355	3.0	0.994	1.508	1.240	0.270	3.0	0.994	1.243	0.010	3.1	0.993
11	2.868	2.688	1.628	1.000	6.6	0.901	2.581	2.718	1.000	6.7	0.898	2.199	0.761	7.3	0.879
13	1.532	0.300	3.000		5.2	0.959	1.956	0.373		5.3	0.957	0.376		5.2	0.958
15	1.079	0.300	3.000	0.998	4.9	0.955	2.182	1.299	1.000	5.0	0.943	0.800	0.601	5.1	0.942
17	1.758	1.791	3.000	0.768	3.9	0.987	1.835	1.779	0.595	3.9	0.988	1.785	0.668	3.9	0.988
19	1.584	1.536	2.734	1.000	3.9	0.990	1.593	1.545	0.845	4.0	0.990	1.577	0.495	4.2	0.987
21	2.996	0.300	3.000	1.000	13.0	0.683	2.995	0.300	1.000	13.0	0.683	0.300	1.000	13.6	0.643
23	0.375	0.300	2.478	0.010	5.1	0.963	0.332	0.300	0.010	5.1	0.962	1.009	0.433	7.2	0.932
25	1.704	1.068	3.000	0.010	3.4	0.983	1.725	1.039	0.124	3.5	0.982	1.075	0.010	3.5	0.983
27	1.837	1.592	2.301	0.249	4.0	0.989	1.797	1.612	0.238	4.0	0.989	1.617	0.252	4.0	0.989
29	1.467	0.597	3.000	0.511	3.7	0.974	1.593	0.697	0.010	3.6	0.975	0.878	0.010	3.7	0.975
31	3.000	1.489	2.733	0.902	5.4	0.975	3.000	1.558	1.000	5.4	0.975	1.156	0.601	6.7	0.962
av					5.4	.947				5.5	.947			5.9	0.940

Column group 3 of Table 5.4 has been adopted as the overall 'best fit'. Compared with the FSR model, which gave average RMS, R^2 and peak error of 7.9, 0.903, and -20.5%, SCHEME has given 5.9, 0.940 and -12.1%. Figures 5.5a-p show the fit obtained for each event with each model. The SCHEME trace (black) generally gives a better match to the variability of the observed hydrograph (blue) - mainly because it treats the urban and rural responses separately rather than as a lumped whole. Events 3 and 21 are poorly fitted by both models, but these are small events and the rainfall seems weakly related to the observed flow (note that the blue and black rainfall traces are for total and effective rainfall). Although SCHEME has performed better, the initial 'spike' on several observed hydrographs is not well predicted. This is discussed further in section 5.4.3.

5.4.2 Discussion of adopted parameter values

The values selected for CHCF and UHUA at stages 2 and 3 of the optimisation in Table 5.4 need some consideration. Firstly, the value 3.0 selected for CHCF is rather higher than the expected value of 1.0 that would yield channel lags equal to the difference in FSR time to peak between their upstream and downstream nodes. Estimating channel lag from Tp has given difficulties in other studies (unpublished), and SCHEME does include alternative procedures based more directly on channel characteristics. However it has not been possible within the time constraints of this study to investigate this further. Table 5.5 gives the length and type of each channel reach included in the study together with the wave speed estimated by SCHEME using CHCF=3.0. Figure 5.1 has shown these reaches schematically, and further information is given in Appendix B. Although some of the wave speeds seem rather high, they are still in

FSR vs SCHEME model, fit events 1,3,5,7)

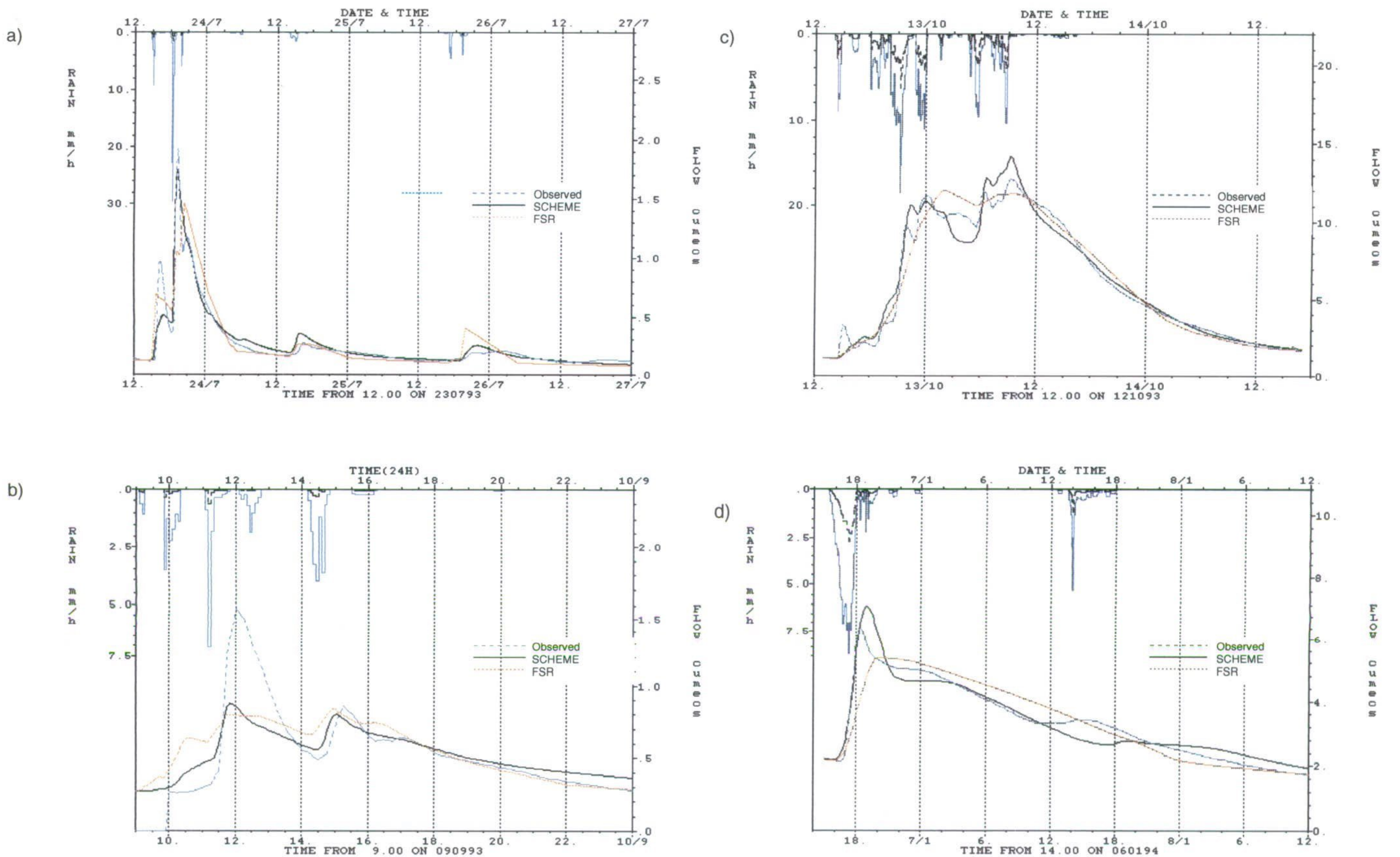


Figure 5.5

FSR vs SCHEME model, fit events 9,11,13,15

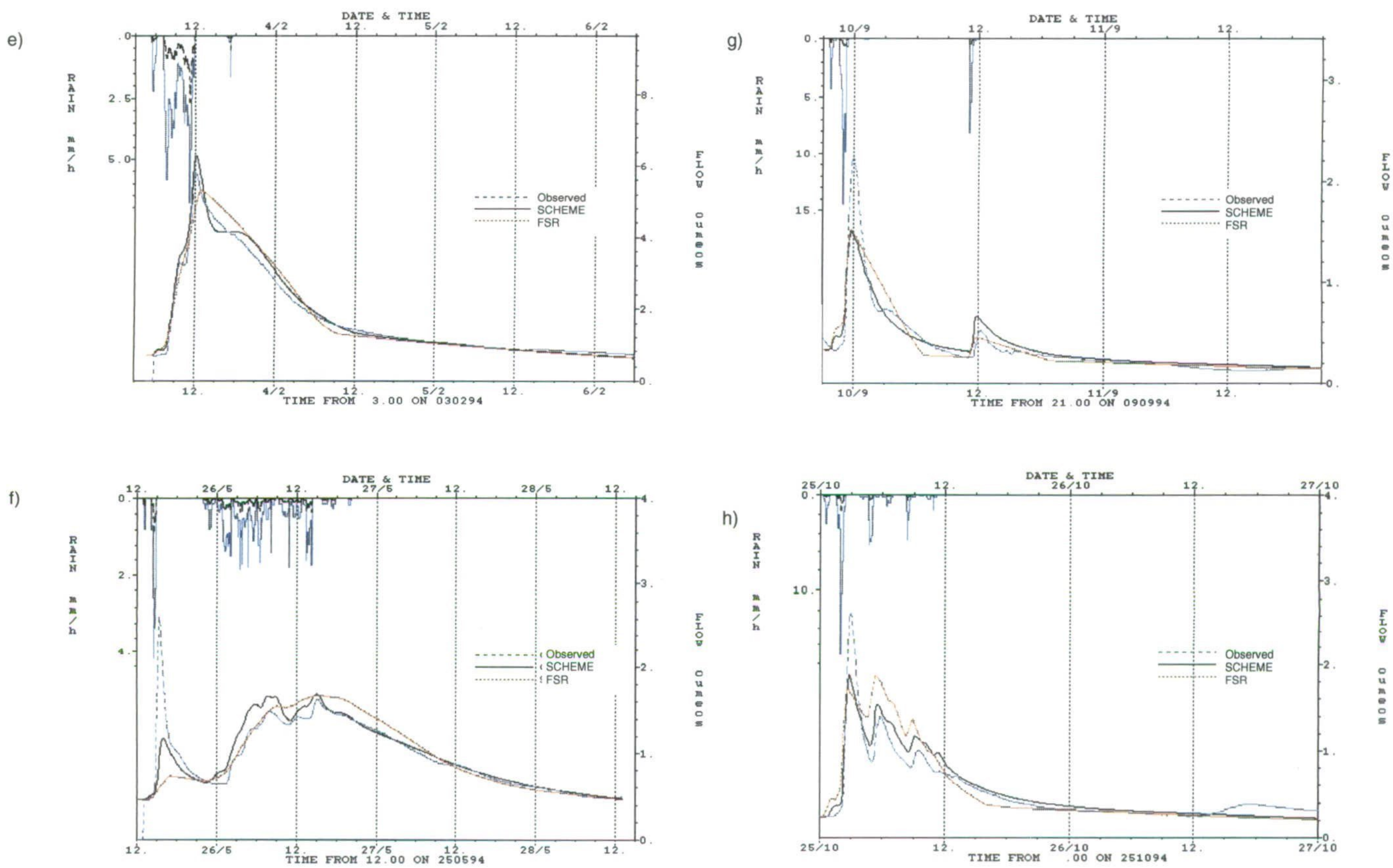


Figure 5.5 cont.

FSR vs SCHEME model, fit events 17,19,21,23

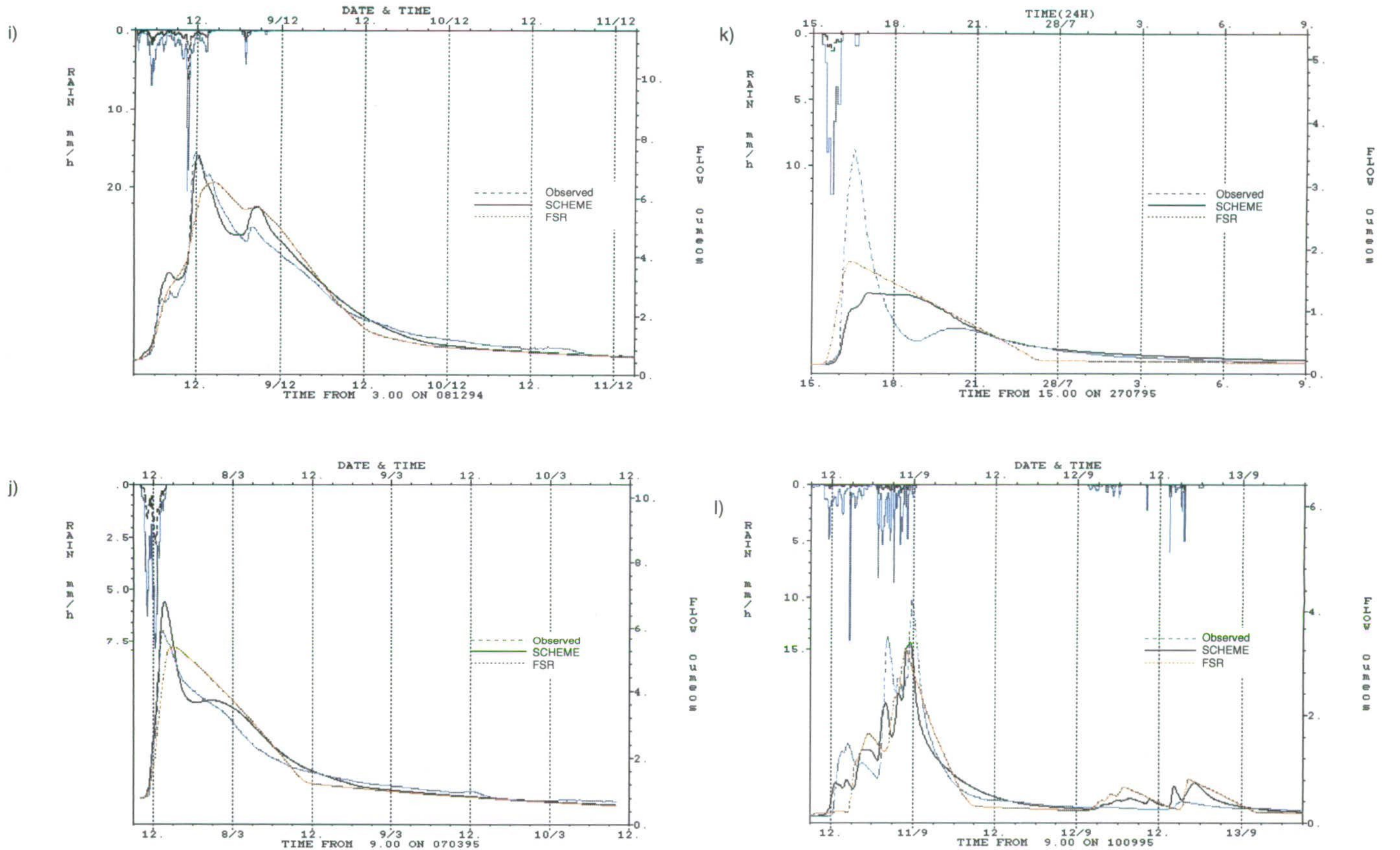
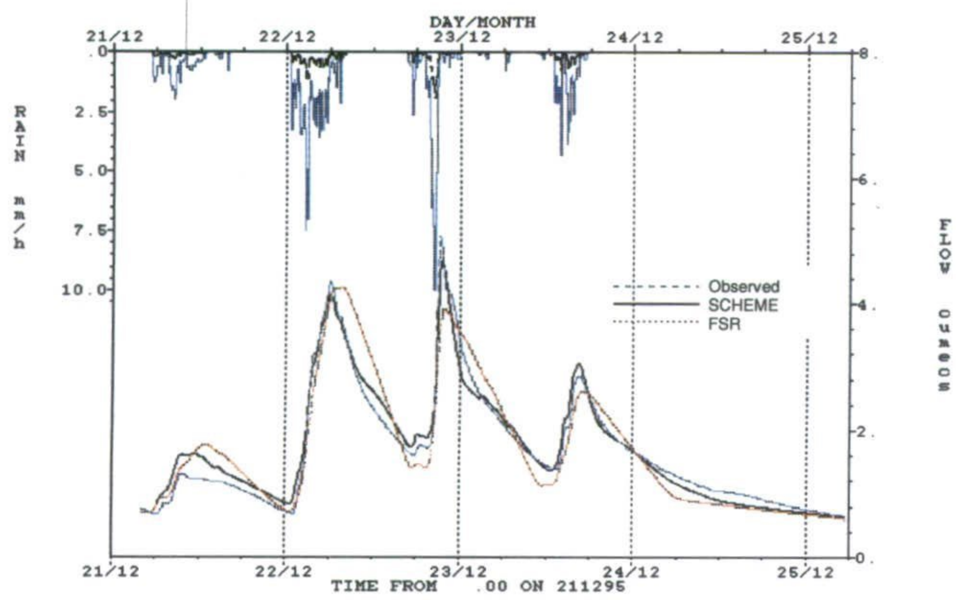


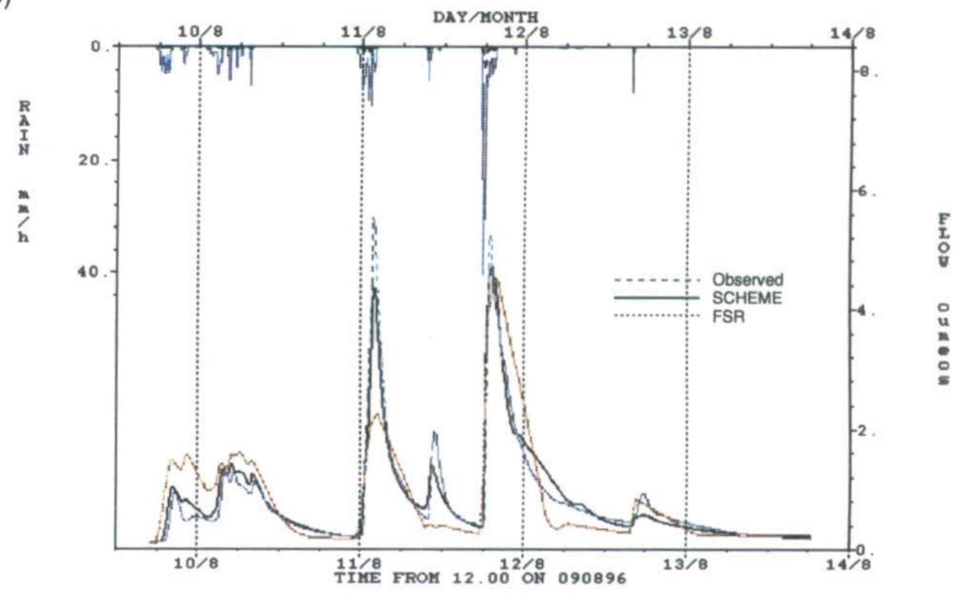
Figure 5.5 cont.

FSR vs SCHEME model, fit events 25,27,29,31

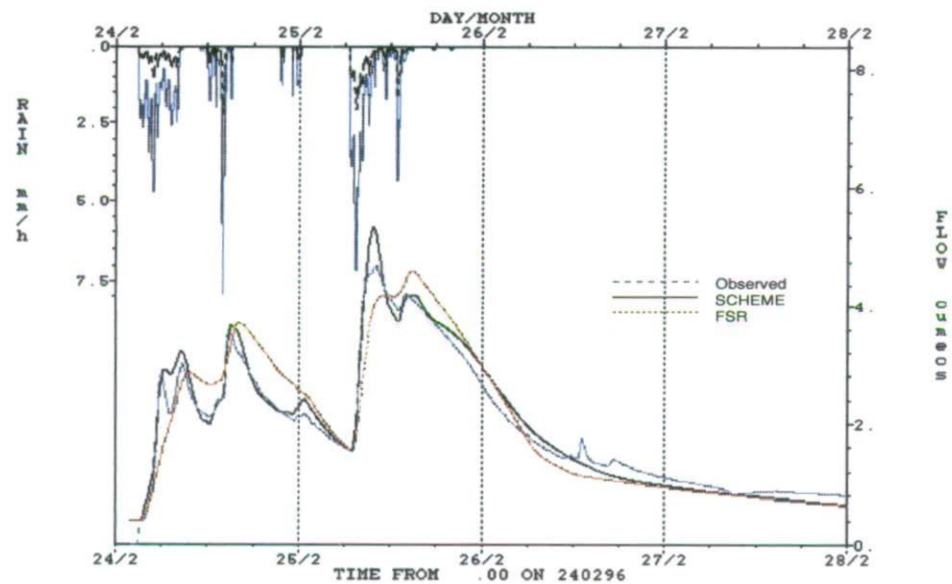
m)



o)



n)



p)

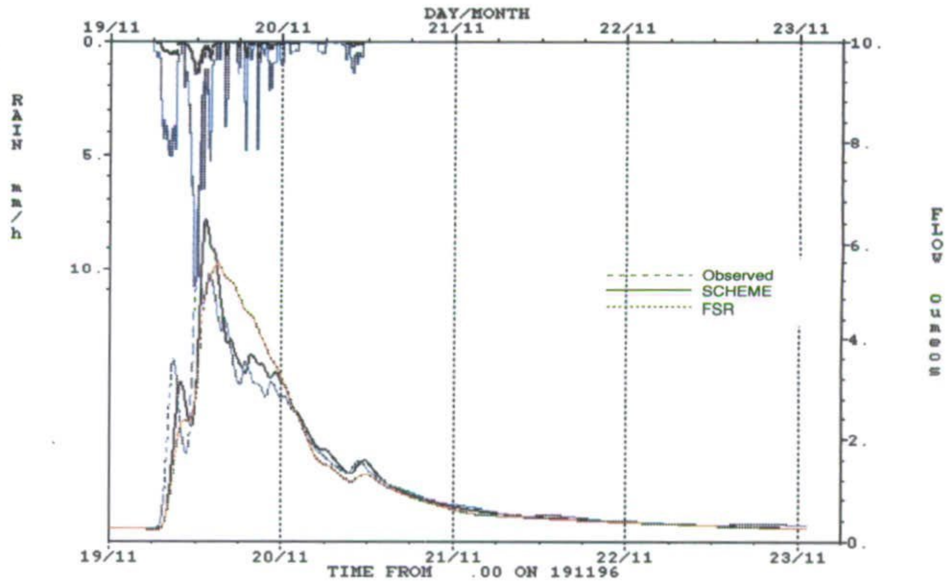


Figure 5.5 cont.

general believable. Analysis of the routing of DWF patterns from Ascot STW to Binfield would give some information on wave speed at relatively low flows, but again this has not been possible within the time constraints of this project.

Table 5.5 Channel and optimised wave speed (m/s) details

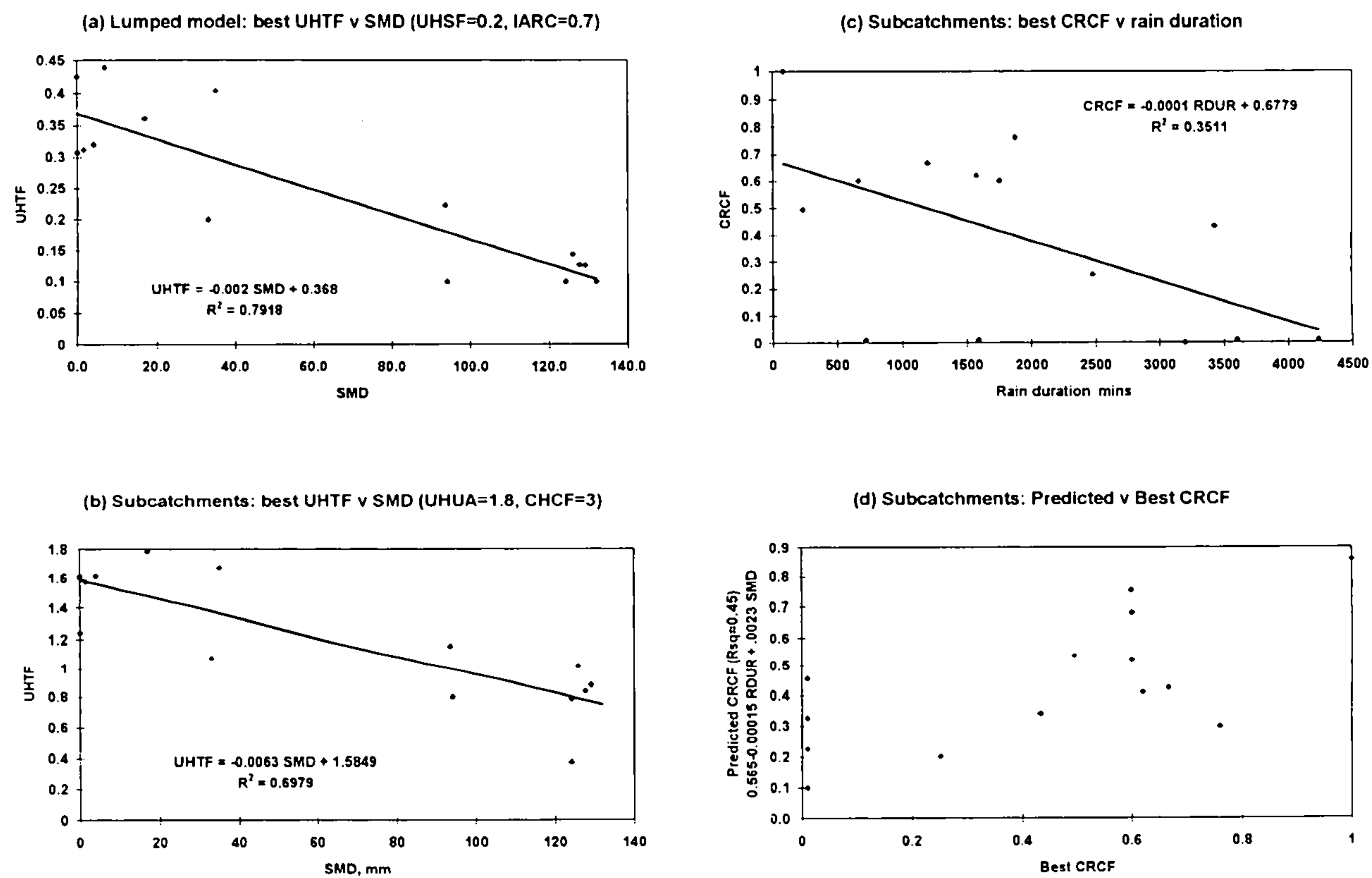
Branch reach	Length (km)	Chanl. type	Wave speed	*Branch .reach*	Length (km)	Chanl. type	Wave speed	*Branch reach*	Length (km)	Chanl. type	Wave speed
1.05	3.3	Cut	2.5	3.55	0.5	Brook	2.8	13.05	2.72	pipe	4.7
1.10	0.8	Cut	2.8	3.65	0.55	Brook	5.1	10.30	0.87	pipe	12.1
3.05	0.4	lined	3.7	1.10	2.3	Cut	4.0	10.35	1.44	pipe	13.3
3.10	1.0	lined	9.3	9.05	0.1	pipe	2.8	17.05	1.15	pipe	3.2
3.20	0.56	pipe	5.2	1.20	0.5	Cut	3.5	18.05	0.75	pipe	4.2
3.35	0.81	pipe	5.6	10.05	0.66	pipe	3.1	17.15	1.12	pipe	5.2
6.05	0.75	pipe	20.8	10.10	0.58	pipe	2.7	19.10	0.46	pipe	3.2
97.10	0.1	pipe	2.8	10.15	0.16	pipe	-	20.10	0.11	pipe	3.1
3.40	1.65	pipe	6.5	10.20	1.56	pipe	5.4	19.15	1.94	pipe	6.0
3.45	1.40	Brook	7.8	11.05	0.37	pipe	-	1.22	1.1	Cut	3.8
8.05	0.2	pipe	0.7	11.10	0.98	pipe	9.1				

Considering next the value of 1.8 for UHUA, this too is greater than might have been expected, given the FSSR16 value of 1.0. However the revised urban adjustment for Tp given by Marshall and Bayliss (1994) also predicts greater impacts than the FSSR16. Moreover, most of the urban subcatchments modelled in this study were 'in sewer', and thus represent conditions not covered by FSSR16, or Marshall and Bayliss. In this context, the high value of UHUA seems reasonable.

Inspection of the 'best fit' UHTF parameters in Table 5.4 indicates that, as with the lumped 'FSR' model in section 5.3, larger values of UHTF are obtained for wetter conditions (low SMD). Although the justification for this is not as clear as in the lumped case (urban and rural differences have already been incorporated in the model), UHTF has again been plotted against SMD, and the resulting relationship (Fig. 5.6b) adopted as the best fit for estimating UHTF for the split record testing.

Inspection of the best fit CRCF values indicates a greater variation than with the 'FSR' model, and some dependence on SMD and storm duration. Figure 5.6c shows the relationship with storm duration alone, explaining 35% of the variance. However, the equation adopted for estimating CRCF in the split record testing includes both storm duration and SMD and explains 45% of the observed variance. The results are illustrated in Fig. 5.6d which shows predicted against optimised CRCF. The parameter equations taken from Fig. 5.6 are therefore:

	UHTF	=	1.585 - 0.0063 SMD	R^2=0.698
	CRCF	=	0.565 - 0.00015 D + 0.0023 SMD	R^2=0.450
where	D	=	Rainfall duration (minutes)	



(a) Lumped model: best UHTF v SMD (UHSF=0.2, IARC=0.7)
UHTF
SMD
UHTF = -0.002 SMD + 0.368
R² = 0.7918
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
(c) Subcatchments: best CRCF v rain duration
CRCF
Rain duration mins
CRCF = -0.0001 RDUR + 0.6779
R² = 0.3511
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
500
1000
1500
2000
2500
3000
3500
4000
4500
(b) Subcatchments: best UHTF v SMD (UHUA=1.8, CHCF=3)
UHTF
SMD, mm
UHTF = -0.0063 SMD + 1.5849
R² = 0.6979
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
20
40
60
80
100
120
140
(d) Subcatchments: Predicted v Best CRCF
Predicted CRCF (Rsq=0.45)
0.565-0.00015 RDUR + .0023 SMD
Best CRCF
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0
0.2
0.4
0.6
0.8
1

Figure 5.6

5.4.3 Model performance for upstream subcatchments

The above discussion has concerned the results for Binfield only, but percentage error in volume (Ve%), and values of R^2 and RMS have been obtained for all gauge locations and are given in Table 5.6. Note first that none of these data have been used in fitting the model, and the remarkably high R^2 values obtained for most events and most gauges indicate that SCHEME's estimation of response time for the various subcatchments is generally good. Some volume errors are however apparent.

Ignoring for the moment shading applied in the table, in every event at Warfield House, and almost every event at Wane Bridge, modelled runoff volume is less than observed. This could be interpreted as model error, but is more likely to be further evidence of the need to revise the ratings obtained for these sites. By contrast, in most of the events at the small rural gauges, Jealotts Ditch and Worldsend, runoff volume has been hugely overestimated. Given that such large errors are not found at Wane Bridge, these results seem to imply that rural runoff is getting into the Cut by a deeper interflow path than is intercepted by the ditches. This needs further investigation, but is more evidence of the difficulty of assessing runoff from small catchments, and thus of estimating the effect of local urbanisation on downstream flows (greater surface runoff but reduced interflow).

Consider now the shading in Table 5.6, and the SMD values that have been copied from Table 5.2 into the final block of Table 5.6. Dark shading indicates dry antecedent conditions (SMD greater than 90 mm), and pale shading indicates wet (SMD less than 5 mm). On examination, it appears that in dry conditions SCHEME has tended to underestimate volumes for the urban catchments (Jocks Lane, Oldbury Bypass, Warfield House), and correspondingly overestimate the rural catchment (Wane Bridge). In wet conditions, the reverse appears to be true. Although it was stated earlier that only Binfield data have been used to optimise the parameter values, a brief attempt was made to correct these volumetric tendencies. In section 3.2 it was mentioned that the Mill Pond inlets had been used to develop the WASSP sewer model. At that time they were estimated as about 40% impervious. On that basis, the default imperviousness used in SCHEME was changed from the FSSR5/16 recommendation of 0.3*URBAN to 0.4*URBAN, and the optimisations were repeated using IARC values of 0.6 and 0.7. This change did improve the volume distribution within the catchment, but in every case increased the model error at Binfield. On balance it was decided the original optimisations given in Tables 5.4 and 5.6 should be retained.

Table 5.6 Best fit at Binfield and corresponding error at upstream gauges

Ev	Wane Bridge			Jealotts Ditch			Worldsend			Warfield House			Benbricke Green		
	Ve%	R^2	RMS	Ve%	R^2	RMS	Ve%	R^2	RMS	Ve%	R^2	RMS	Ve%	R^2	RMS
1	4.6	0.256	28.9							-71.5	0.771	11.2	18.3	0.569	7.5
3	-3.7	0.248	22.1							-78.6	0.325	40.7	-19.6	0.596	9.4
5	-59.8	0.960	36.3							-46.5	0.956	19.9	23.1	0.670	16.7
7	-66.8	0.685	44.4										7.1	0.919	7.2
9	-16.8	0.883	19.2										31.3	0.935	5.2
11	-40.8	0.945	26.6							-60.0	0.909	18.5	-25.4	0.328	13.0
13	12.5	-.641	123.							-73.0	0.533	15.1	-6.7	0.862	6.3
15	-40.4	0.310	41.1							-79.7	0.721	25.8	39.7	0.821	6.0
17	-34.0	0.936	18.4							-2.5	0.952	4.6	17.7	0.848	6.9
19							-15.7	0.782	15.8	-22.9	0.979	6.7	67.5	0.937	11.4
21	570.0	-.144	739.							-42.8	0.182	20.9	-24.7	0.825	7.4
23	178.3	0.423	157.							-42.9	0.707	21.2	14.9	0.805	8.8
25	-2.7	0.890	11.1	110.2	0.889	37.9	1796.	0.324	49.7	-31.8	0.962	7.5	57.3	0.896	9.3
27	-11.1	0.898	12.9	11.10	0.653	12.6	1189.	0.882	590.	-6.9	0.959	6.3			
29	128.3	0.695	36.3	2858.	0.365	430.				-44.0	0.388	16.6			
31	-32.5	0.985	10.9	434.0	0.900	135.	138.0	0.756	58.0	-72.0	0.850	23.2			

Ev	Easthampstead			Great Hollands			Wildridings			Mill Pond Outfall			Waitrose		
1	-25.5	0.597	7.7	91.9	0.578	6.5	-63.5	0.902	2.8	216.5	0.714	16.3	-22.9	0.628	9.6
3	-39.8	0.432	13.5	51.1	0.390	11.8	-48.7	0.347	10.4	60.4	0.752	16.8	-43.3	0.699	9.9
5	34.1	0.857	11.4	83.2	0.715	16.3	10.5	0.806	9.5	170.1	0.913	57.8	387.0	0.582	44.5
7				40.2	0.793	11.0	-29.3	0.943	5.5	188.0	0.880	29.2	472.0	0.920	25.2
9				4.5	0.925	8.6				166.0	0.970	19.6	932.0	0.925	38.5
11	96.9	0.492	9.3	-10.9	0.693	6.0				9.2	0.722	8.8	-10.2	0.218	14.4
13	41.3	0.844	4.2	12.1	0.774	5.7				96.5	0.941	6.5	-38.8	0.788	9.6
15	100.5	0.942	8.3	125.0	0.854	5.2				2276.	0.604	54.6	119.0	0.464	6.2
17	146.0	0.866	10.8	109.5	0.781	8.6				136.5	0.975	18.1	56.1	0.758	4.9
19	155.0	0.948	10.1	295.0	0.863	12.1				132.0	0.980	19.9	90.9	0.911	5.1
21	67.2	0.863	6.8							12.0	0.943	6.7	-33.0	0.507	7.3
23	72.2	0.398	17.7							78.3	0.641	17.4	-23.5	0.641	7.1
25	36.1	0.918	4.8							87.8	0.957	13.9	42.6	0.609	6.1
27	186.0	0.882	19.7	63.0	0.831	11.5							52.5	0.843	6.0
29	-78.0	0.841	10.8	-12.3	0.826	3.9							-18.7	0.670	4.2
31	-29.0	0.955	6.5	10.1	0.929	6.7	-39.0	0.953	5.0	766.0	0.916	81.0	236.0	0.960	16.2

Ev	Industrial			Oldbury Bypass			Jocks Lane			Binfield Weir			SMD (mm)
1	22.6	0.387	10.1	-37.4	0.752	6.6	-24.4	0.892	4.0	2.9	0.966	3.5	127.8
3	41.0	0.192	11.6	-41.8	0.807	9.3	-39.9	0.882	11.7	0.4	0.905	12.1	124.3
5	97.8	0.629	12.8	-4.1	0.729	11.3	-2.4	0.933	7.4	0.4	0.983	5.6	35.1
7	662.0	0.736	27.5	3.3	0.891	6.4	10.5	0.936	5.3	0.7	0.953	5.6	0.0
9	615.0	0.847	24.7	26.0	0.948	4.0	9.6	0.989	2.9	2.9	0.993	3.1	0.0
11	43.2	0.209	14.4	33.4	0.295	13.2	-0.2	0.619	5.5	2.3	0.879	7.3	6.9
13	196.0	0.682	15.0	-46.0	0.888	9.2	-33.2	0.945	5.9	4.1	0.958	5.2	124.2
15	57.6	0.611	6.3	-15.9	0.859	5.8	-19.3	0.802	5.4	3.3	0.942	5.1	94.1
17	132.0	0.605	6.7	-1.4	0.820	4.9	1.5	0.944	4.2	1.0	0.988	3.9	17.0
19	139.0	0.819	8.6	28.9	0.933	4.1	10.5	0.987	2.5	2.3	0.987	4.2	1.5
21	-39.5	0.982	6.2	-56.2	0.887	8.4	-62.8	0.709	10.3	-3.2	0.643	13.6	132.1
23	78.6	0.687	5.1	-24.9	0.677	8.3				-1.1	0.932	7.2	126.1
25	41.3	0.723	6.7	-14.2	0.857	5.6	-9.8	0.973	4.1	1.7	0.983	3.5	33.0
27	77.7	0.771	11.5	18.5	0.878	5.8	21.3	0.973	6.9	1.5	0.989	4.0	4.0
29	28.6	0.400	4.9	-16.7	0.827	5.6	-9.3	0.926	4.1	0.2	0.975	3.7	129.3
31	69.7	0.901	6.0	-16.6	0.959	4.3	-19.9	0.848	10.4	0.8	0.962	6.7	93.6

Figures 5.7 to 5.12 show sample plots of observed and predicted hydrographs at six gauges upstream of Binfield (the same sample events as used in Fig. 5.2). These represent the two largest fit events as measured at Binfield (nos 5 and 17), one small event (no 11) and one complex event (no 25).

Figure 5.7 is for Wane Bridge, showing the lack of fit is largely related to underestimation of both surface runoff and baseflow volumes, though the underprediction of the first and overprediction of the third peak in Fig. 5.7d (event 25) does suggest a higher CRCF might be more appropriate (but the fit at Binfield, Fig. 5.5m, was near perfect). Figure 5.8 for Warfield House generally suggests a good fit, but with the same concerns for surface and baseflow volume estimation. However, the uncertainty concerning the ratings for these two sites must be borne in mind. Figure 5.9 for Jocks Lane shows a remarkably good fit, though the initial 'spike' is less well predicted. What has caused the spike is not clear, it could be kinematic effects in the pipe system, rainfall direct onto reservoir surfaces (not modelled in SCHEME), or perhaps poor estimation of rainfall. Figures 5.10 to 5.12 show the fit in some smaller urban catchments. In each case, although the fit is generally good, there is a suggestion that the model is a little slow. Note that the observed data for event 11 at Benbricke Green and Oldbury have not been fully abstracted, as only the first peak had seemed significant when these data were first examined (in isolation). This accounts for the low R^2 values given in Table 5.6. Note too the initial 'spike' is often present in the response at these gauges, which suggests direct rainfall onto a reservoir is not the cause - the catchments to neither Benbricke Green nor Oldbury contain 'wet' ponds . Finally, on Fig. 5.11a, note the throttled outflow from the Mill Pond has been predicted, but at a higher discharge than observed.

5.5 SPLIT RECORD TESTS

With the FSR and detailed SCHEME models fitted as described, and with the resulting relationships determined from the fit events allowing prediction of parameter values (Recharge Factor and PR from Fig. 5.3, UHTF and CRCF from Fig. 5.6), the test events could be modelled. As a final preliminary, however, the fit events were modelled using predicted parameter values to assess the degradation in fit involved, and provide a true comparison for the test events. The results are given in Table 5.7, which also compare the average error values from the predicted parameters with the 'best fit' values from sections 5.3 and 5.4 (added as the last line of Table 5.7). This gives a better indication of the likely error that may be expected with the test events.

Table 5.8 shows the 'non-fixed' parameter values for both FSR and SCHEME models, together with the corresponding error measures obtained for the test storms (note, SMD for these events was given in Table 3.3).

A summary of the average errors from Tables 5.7 and 5.8 is given in Table 5.9, showing that in terms of R^2 (describing the fit to the general shape of the hydrograph) SCHEME has performed considerably better than the lumped FSR model. Note also that SCHEME has estimated hydrograph shape for the test storms slightly better than for the fit storms, though the FSR method has performed worse. Neither model has predicted volume well for the test storms, both showing a 20% overestimate compared with 5% with the fit storms. Despite this, SCHEME's advantage in terms of RMS and peak error has been maintained, and its average estimation of peak has actually improved.

Sample events at Wane Bridge (5,11,17 and 25)

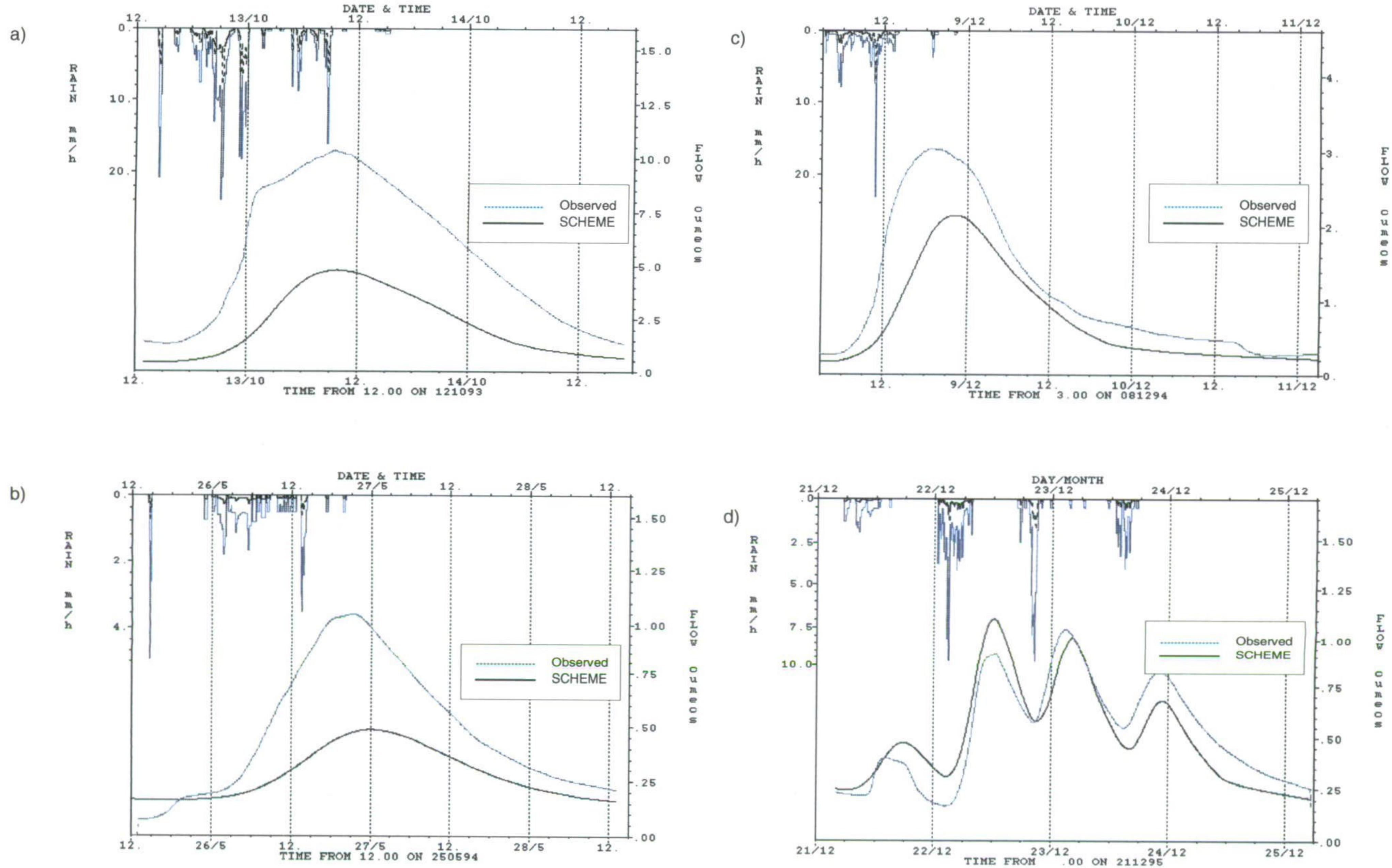


Figure 5.7

Sample events at Warfield House (5,11,17 and 25)

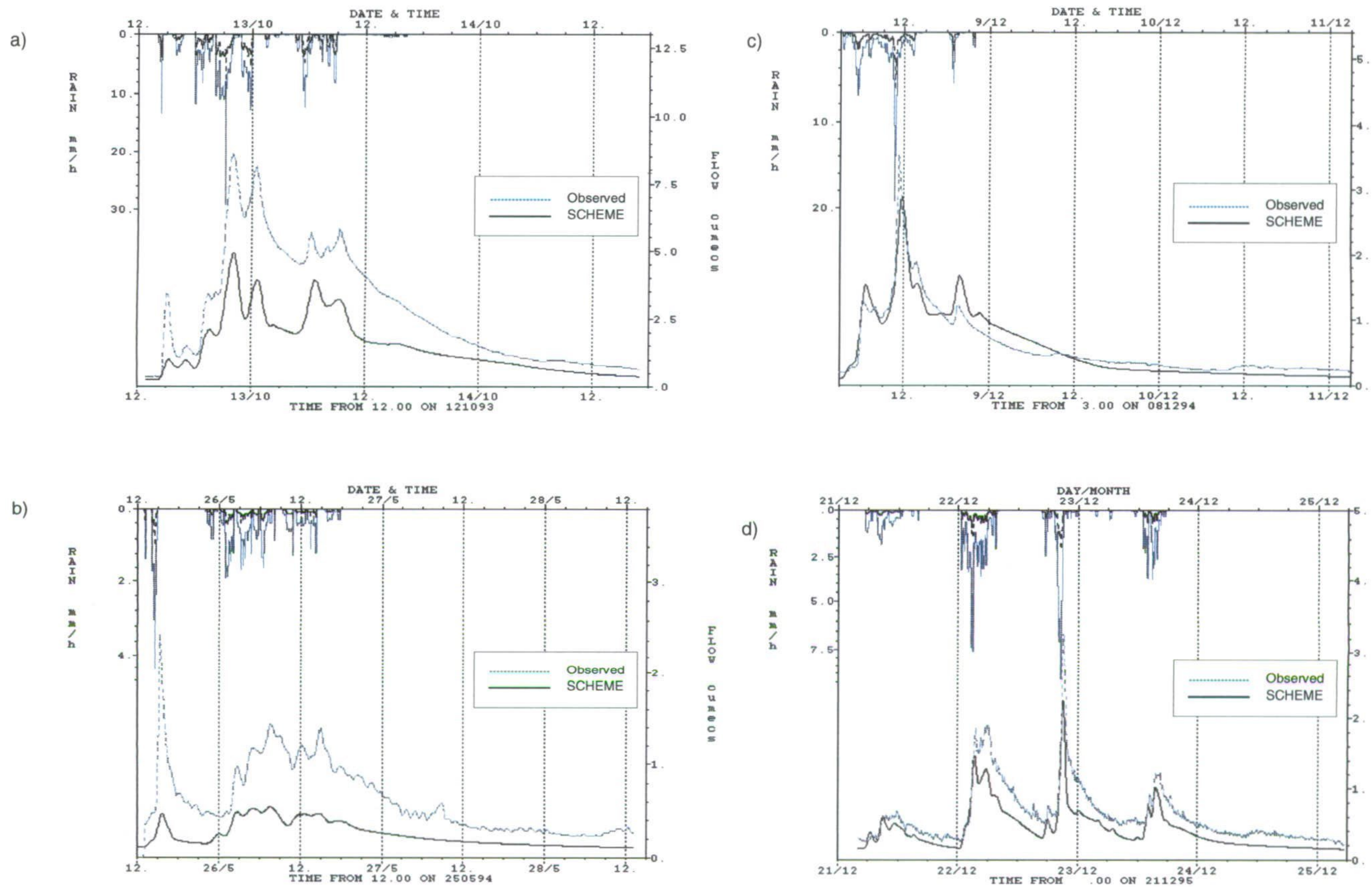


Figure 5.8

Sample events at Jocks Lane (5,11,17 and 25)

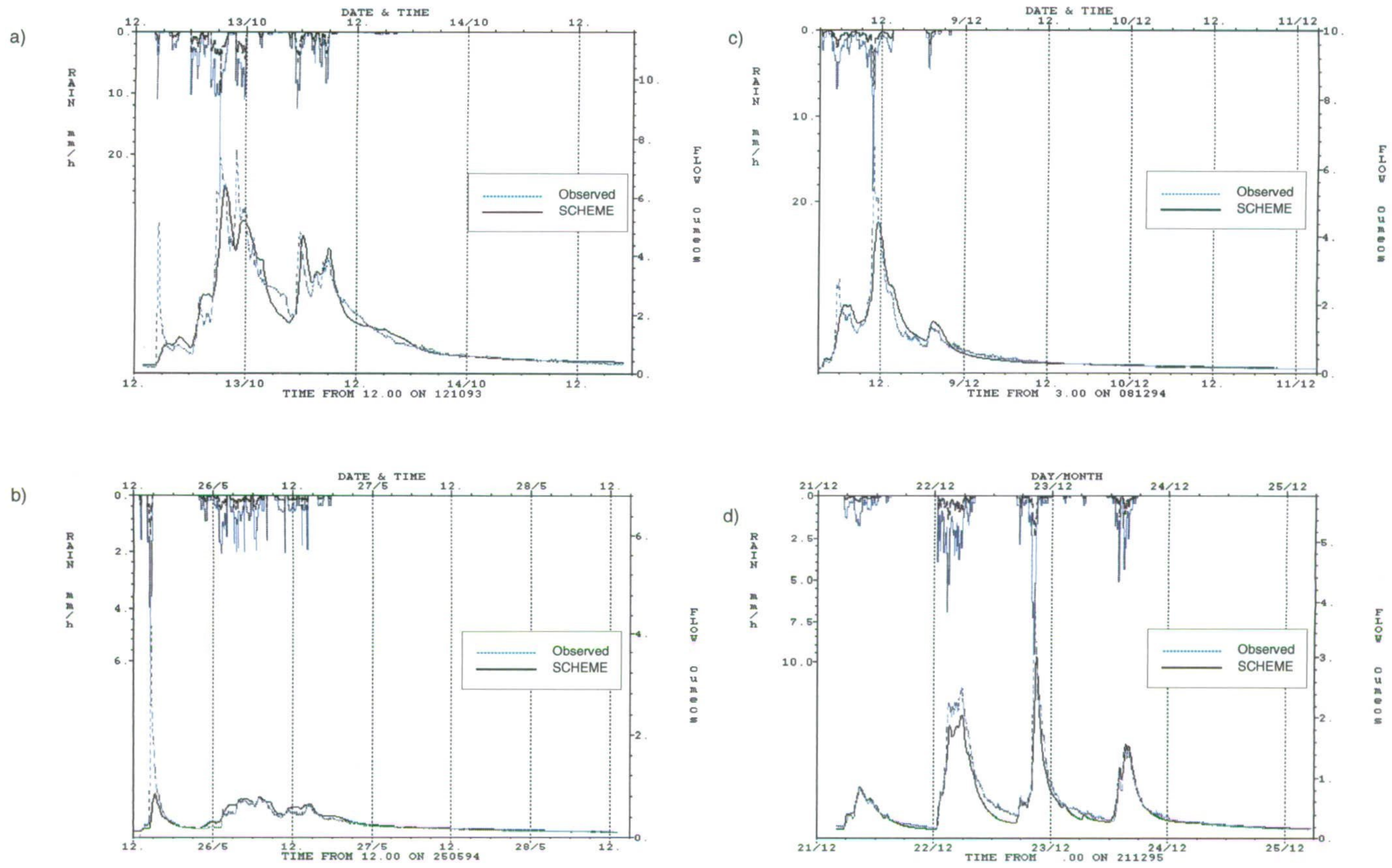


Figure 5.9

Sample events at Benbricke Green (5,7,11 and 25)

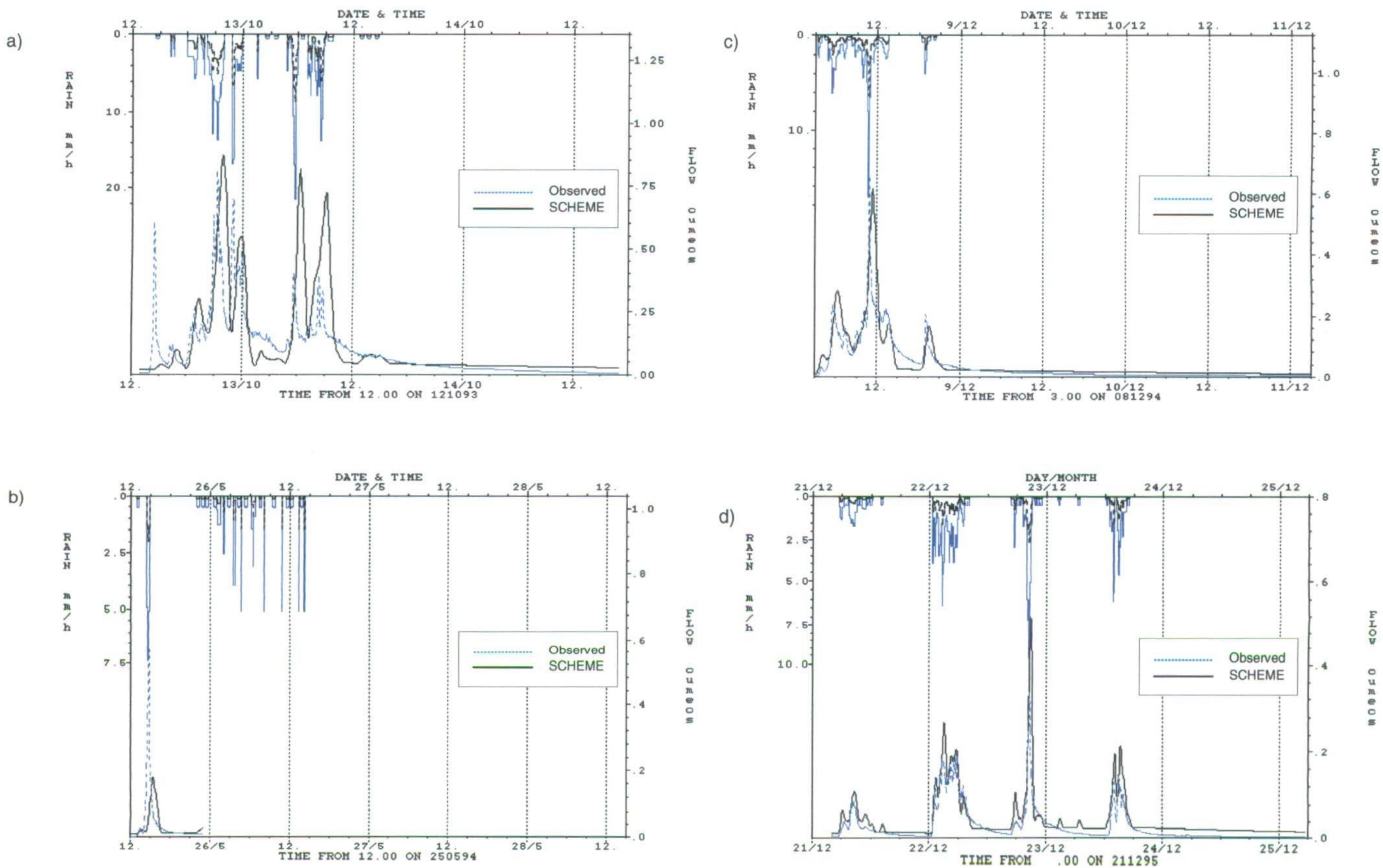


Figure 5.10

Sample events at Mill Pond outfall (5,11,17 and 25)

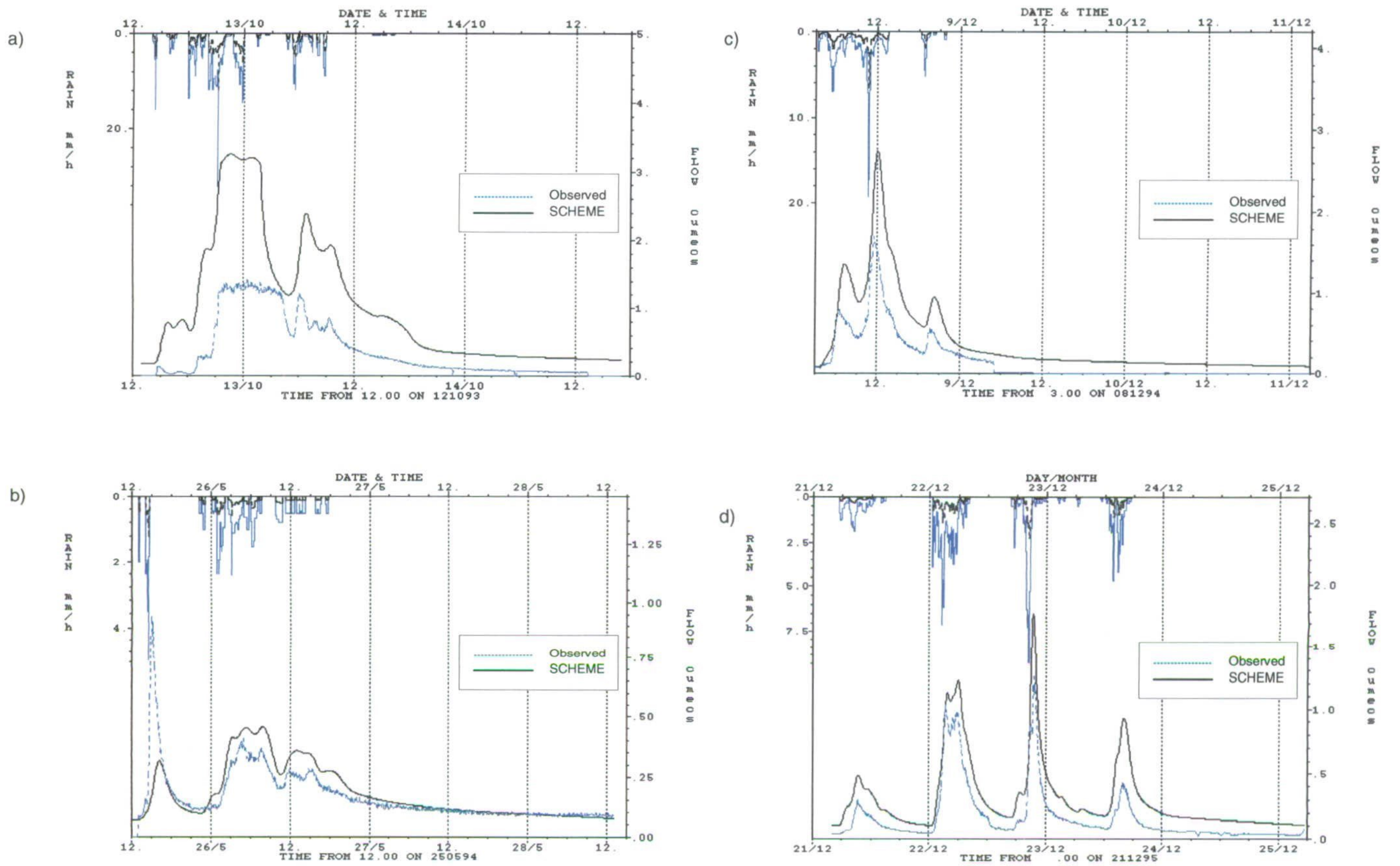


Figure 5.11

Sample events at Oldbury Bypass (5,11,17 and 25)

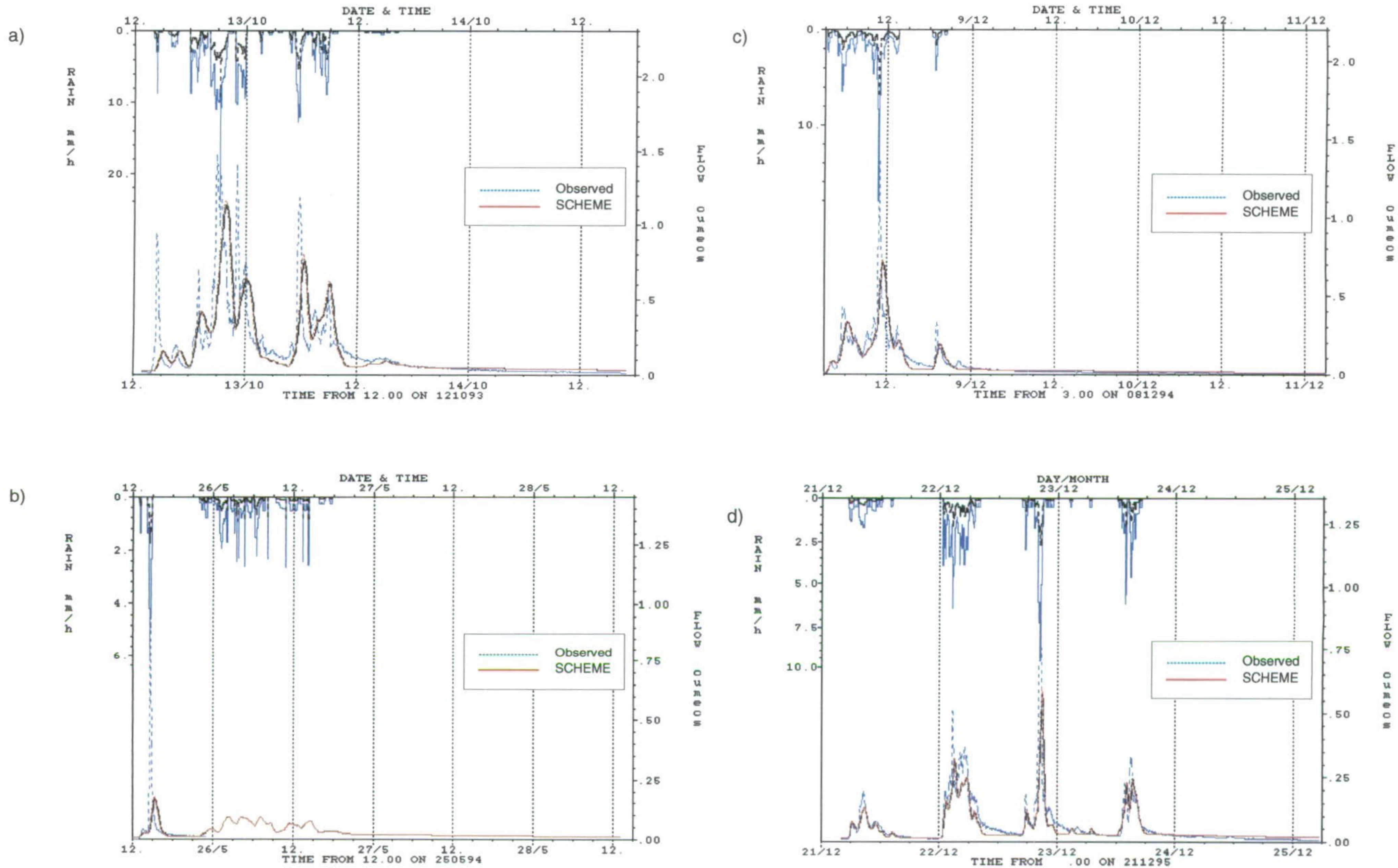


Figure 5.12

Table 5.7 Fit events, predicted parameters

Ev	QP (m^3/s)	Recharge	'FSR' model						SCHEME						
			PR	UHTF	Ve%	R^2	RMS	Qpe%	PR	UHTF	CRCF	Ve%	R^2	RMS	Qpe%
1	1.93	0.644	0.076	0.112	14.8	0.914	8	-2	0.074	0.780	0.379	22.6	0.964	5.6	7.2
3	1.58	0.655	0.082	0.119	25.1	0.742	15.9	-34.6	0.080	0.802	0.751	2.5	0.907	11.7	-42.9
5	12.77	0.934	0.225	0.298	-22	0.961	14.3	-19.5	0.224	1.364	0.409	-19.2	0.974	13.3	-13.9
7	6.44	1.044	0.281	0.368	1.4	0.942	8.7	-5.7	0.280	1.585	0.327	3	0.953	6.3	17
9	5.83	1.044	0.281	0.368	1.2	0.969	5.6	-16.6	0.280	1.585	0.457	0	0.981	4.3	0.6
11	2.63	1.022	0.270	0.354	18.8	0.818	15.9	-14.2	0.269	1.542	0.299	17	0.873	13	-11.2
13	2.23	0.655	0.082	0.120	33.3	0.922	8.5	-16.2	0.080	0.803	0.701	33.2	0.941	7.3	-26.5
15	2.64	0.749	0.130	0.180	28.8	0.797	14.6	-25.8	0.129	0.992	0.682	26	0.91	9.7	-18.9
17	7.53	0.991	0.254	0.334	1	0.974	5.4	-17.7	0.253	1.478	0.424	0.5	0.992	3.1	-4.3
19	5.92	1.039	0.278	0.365	-10.8	0.94	7.8	-31.3	0.278	1.576	0.534	-7.6	0.987	4	3.5
21	3.58	0.631	0.069	0.104	-38.9	0.792	14	-73.6	0.067	0.753	0.857	-32	0.662	15.5	-16.7
23	4.23	0.649	0.079	0.116	-6.7	0.843	9.9	-28.2	0.077	0.791	0.340	-14	0.922	8.5	-29.2
25	5.08	0.941	0.228	0.302	32.4	0.912	13.8	-11.3	0.227	1.377	0.100	29.4	0.977	11.1	12.8
27	4.74	1.031	0.274	0.360	8.9	0.916	10.8	-12.1	0.274	1.560	0.202	7.5	0.99	5.4	17.7
29	5.51	0.639	0.074	0.109	-2.4	0.843	8.9	-18	0.072	0.770	0.227	-7.3	0.974	4.4	-21.1
31	5.45	0.751	0.131	0.181	12.7	0.946	10.8	24.3	0.129	0.995	0.517	8.1	0.963	7.1	23.5
av					6.1	0.889	10.8	-18.9				4.4	0.936	8.1	-6.4
best fit					-	0.903	7.9	-20.5				-	0.940	5.9	-12.1

Table 5.8 Test events, predicted parameters

Ev	QP (m^3/s)	Recharge	'FSR' model						SCHEME						
			PR	UHTF	Ve%	R^2	RMS	Qpe%	PR	UHTF	CRCF	Ve%	R^2	RMS	Qpe%
2	2.44	0.622	0.065	0.099	5.9	0.937	10.6	-2.7	0.063	0.736	0.805	0.3	0.986	4.1	7.8
4	4.53	0.892	0.203	0.271	39.8	0.727	19	-25.5	0.202	1.279	0.610	43.6	0.967	16.2	22.9
6	5.18	1.044	0.281	0.368	2.6	0.969	6.2	-5.5	0.280	1.585	0.335	-1.7	0.974	5.5	16.3
8	9.31	1.044	0.281	0.368	-23.6	0.83	15.7	-45.8	0.280	1.585	0.228	-18.5	0.922	12.3	-36.7
10	4.51	1.044	0.281	0.368	6	0.907	8.6	-15.4	0.280	1.585	0.204	8.5	0.976	6.1	21.3
12	4.16	0.857	0.185	0.249	147.	0.263	59.8	11.2	0.184	1.210	0.662	96.2	0.83	36.4	29.3
14	5.64	0.696	0.103	0.145	40	0.943	15.9	1.6	0.101	0.884	0.730	37.7	0.948	13	21.7
16	6.83	0.890	0.202	0.269	22.6	0.957	10.9	7.1	0.201	1.274	0.435	21.4	0.996	6.8	9.1
18	6.48	1.044	0.281	0.368	-2.5	0.943	7.4	-30.8	0.280	1.585	0.312	2.4	0.98	4.6	-2
20	1.96	0.627	0.068	0.102	-5.7	0.825	10.9	-34.9	0.066	0.746	0.820	4.2	0.96	5.2	-14.6
22	4.19	0.619	0.064	0.097	-26	0.65	11.9	-81.1	0.062	0.730	0.816	-4.4	0.823	9.7	-66
24	6.01	0.873	0.194	0.259	56.2	0.907	27.4	22.1	0.192	1.242	0.536	55.3	0.954	23.4	27.1
26	5.57	0.991	0.254	0.334	8.5	0.985	5.4	-3.3	0.253	1.479	0.284	10	0.996	4.4	7.6
28	3.24	0.711	0.111	0.155	79.6	0.797	2.79	-13.8	0.109	0.915	0.531	57	0.898	10.6	-7.9
30	3.75	0.650	0.079	0.116	0	0.915	8.3	-9.4	0.078	0.792	0.298	-1.2	0.922	7.4	-14.4
av					23.3	0.837	14.7	-15.1				20.7	0.942	11.0	1.4

Table 5.9 Summary of fit/error criteria, FSR and SCHEME, fit and test events

		FSR				SCHEME			
Parameters,	**Events**	**Ve%**	**R^2**	**RMS**	**Qpe%**	**Ve%**	**R^2**	**RMS**	**Qpe%**
Best fit	fit	-	0.903	7.9	-20.5	-	0.940	5.9	-12.1
Predicted	fit	6.1	0.889	10.8	-18.9	4.4	0.936	8.1	-6.4
Predicted	test	23.3	0.837	14.7	-15.1	20.7	0.942	11.0	1.4

The fit of both models is also shown for all the test storms in Fig. 5.13. Inspection shows the effect of volume errors more clearly, with (d)=event 8, (f)=event 12, and (l)=event 24 otherwise well modelled in terms of shape (event 12 seems to be a baseflow problem, the others mainly direct runoff). Poor volume estimation was not totally unexpected; it has long been recognised (see FSR) as the more difficult aspect of flood modelling to estimate accurately.

The good fit to hydrograph shape shown by the SCHEME model, reproducing the pulses of peak flow compared with the more smoothed response of the FSR model, is particularly noteworthy. Within Tables 5.4 and 5.6 (best-fit parameters) and Tables 5.7 and 5.8 (predicted parameters) SCHEME has consistently improved over the lumped FSR model, and this improvement has not been swamped by problems of volume prediction. Together with the ability to predict hydrographs throughout the catchments based on measured performance downstream, these results demonstrate that SCHEME is a significant improvement for modelling mixed urban/rural catchments, and forms a credible basis for estimating the effect of networks of flood storage ponds.

5.6 STORAGE PONDS

Previous sections have demonstrated that SCHEME is able to represent the variations in phasing and scale of response from the various subcatchments in a mixed urban/rural catchment, and thus to account for local drainage throttles and flood storage ponds. The model may thus be used to investigate in broad terms the effect of the 18 flood storages included in this study.

Table 5.10 gives the ratio of estimated peak outflow to inflow at each pond for each of the fit events, based on the 'best-fit' parameters. This table shows that the off-line ponds (The Warren, Bay Road, South Hill Park 3, Oldbury) have had a relatively small impact, except for South Hill Park 3 in the larger events (5,17). The large ponds (Savernake, Jiggs Lane, Mill Pond, Waterside Park) generally have the greater effect, and it may be noted that Jiggs Lane and Waterside Park are particularly large in relation to their catchment area. The ornamental lakes (Ascot Place, Warfield House, Binfield Lake) have a reasonable overall impact, but a smaller effect on the larger events.

Test events(2,4,6,8,10,12,14, and 16)

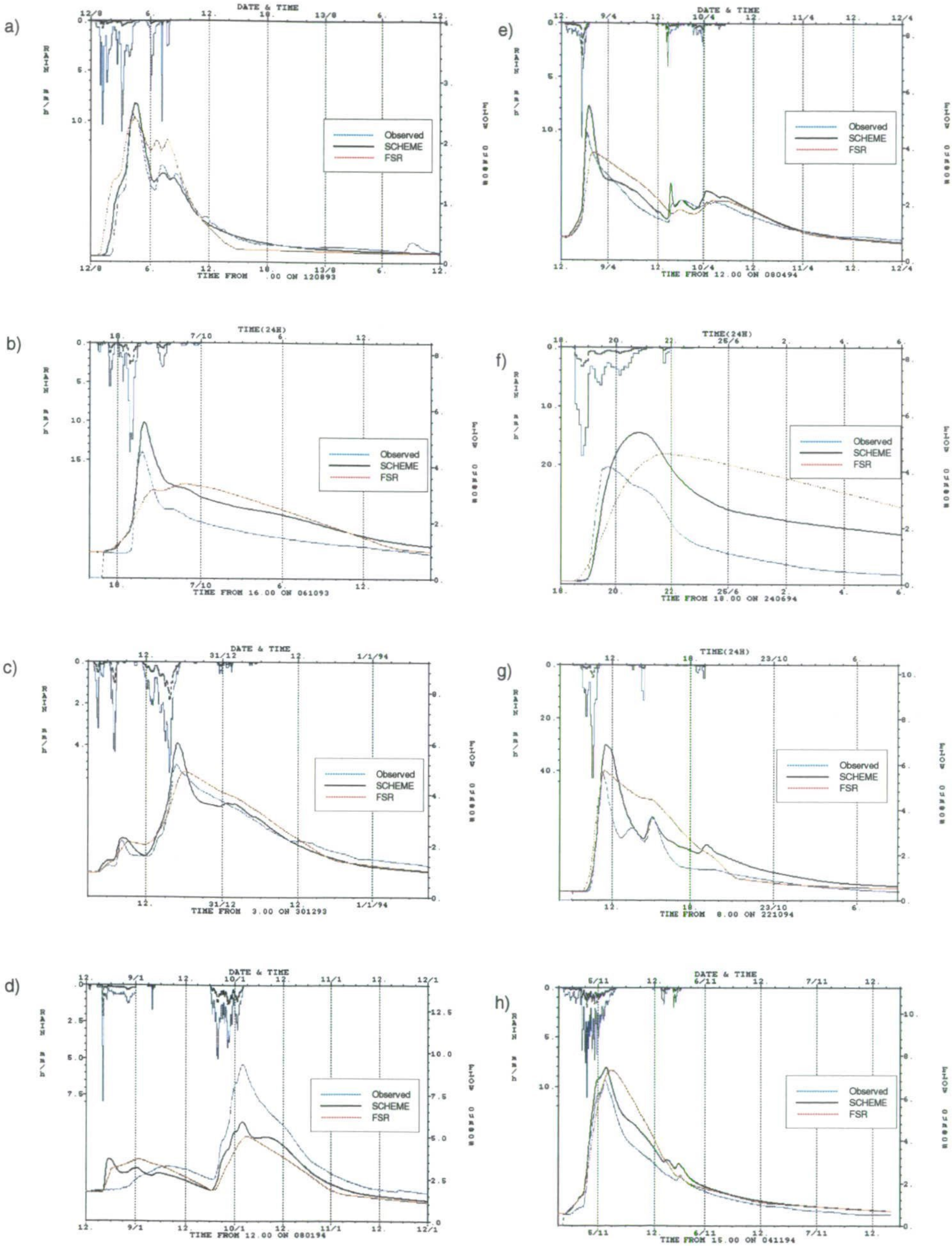


Figure 5.13

Test events(18,20,22,24,26,28 and 30)

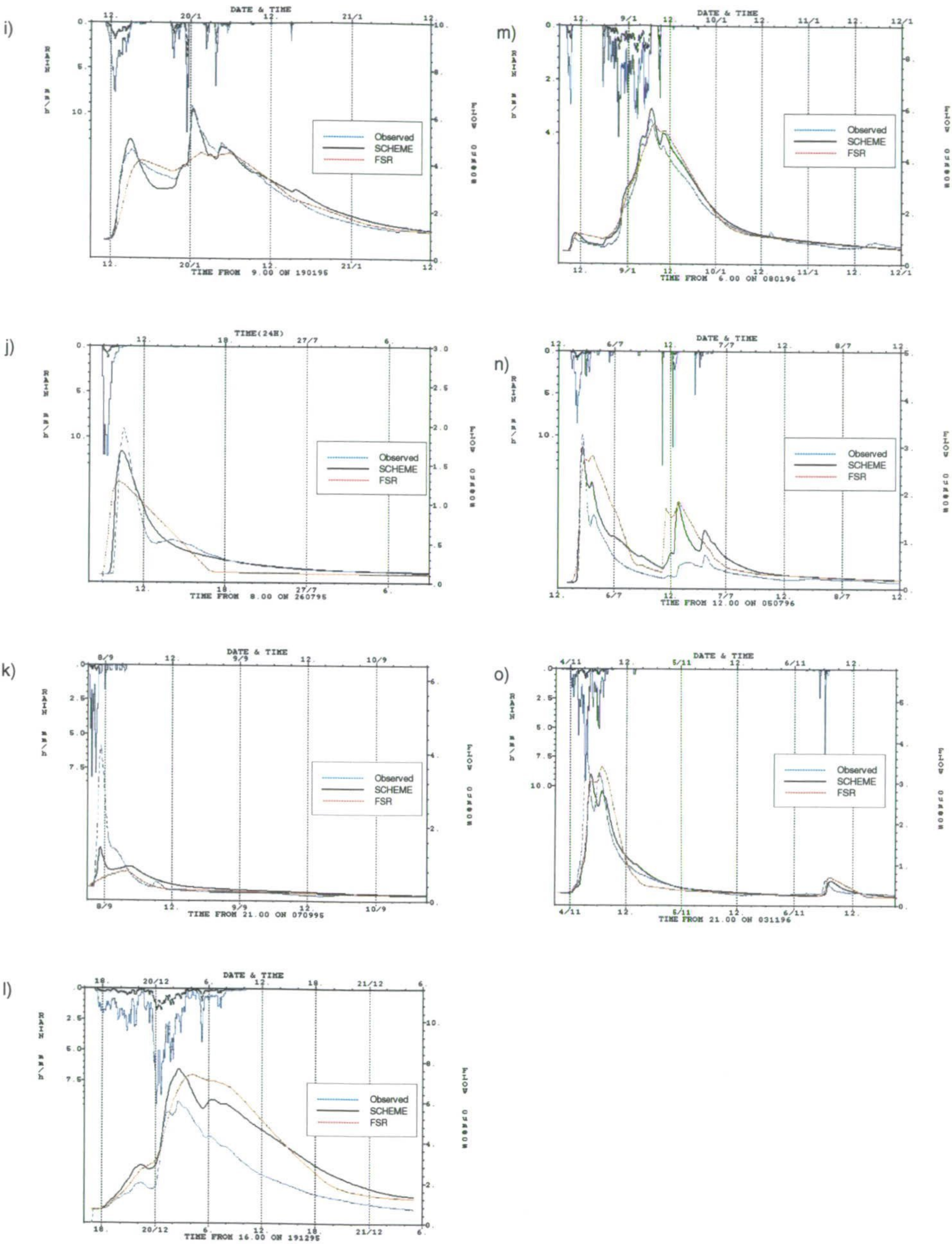


Figure 5.13

Table 5.10 Ratio of Peak outflow to inflow (as %) at storage ponds, fit events

Ev	Ascot Place	Saver-nake	The Warren	Martins Heron	Bay Road	Jiggs Lane	Warfield House	S.Hill Park 1	S.Hill Park 2
1	43	47	99	55	99	12	55	51	73
3	74	48	100	46	99	29	55	57	69
5	92	64	99	88	100	19	83	94	56
7	84	29	74	88	100	24	78	96	69
9	76	48	100	87	100	31	78	90	93
11	93	93	100	83	100	59	94	96	96
13	42	23	98	37	98	15	29	50	62
15	63	32	98	45	99	15	36	61	72
17	86	49	99	83	100	26	80	93	84
19	77	40	100	87	100	27	77	93	95
21	22	17	96	35	99	8	22	50	53
23	74	58	99	62	100	32	71	66	85
25	74	39	100	73	100	22	62	90	91
27	87	52	100	81	100	34	75	96	95
29	54	27	98	62	99	13	48	73	83
31	73	41	100	80	100	27	72	93	95
Av	70	44	98	68	100	25	63	78	79

Ev	S.Hill Park 3	Sports Centre	Mill Pond	Oldbury Pond	Amen Corner	Water-side Park	St Johns Ambulce.	M.Storey Car Park	Binfield Lake
1	99	86	44	90	61	38	50	52	67
3	100	87	52	87	72	39	90	97	65
5	65	100	65	98	97	51	62	68	99
7	61	100	85	99	91	55	88	90	99
9	64	99	83	96	91	48	70	69	96
11	100	99	88	97	96	77	99	98	98
13	100	86	47	92	64	29	79	78	76
15	100	90	48	90	58	38	68	65	67
17	59	99	87	99	94	50	63	67	98
19	60	100	87	98	90	41	73	77	97
21	99	75	26	89	34	18	70	65	56
23	100	97	76	94	90	62	66	64	92
25	81	94	70	97	81	47	77	75	93
27	93	97	84	99	92	50	76	77	99
29	58	96	55	91	66	37	41	44	84
31	62	99	81	98	89	48	79	81	97
Av	81	94	67	95	79	46	72	73	86

Considering the largest event (5) in greater detail, Table 5.11 shows the maximum volumes stored at each pond. Comparing this table with the peak reductions given Table 5.10 confirms that the available storage in the ornamental lakes is not well utilised in terms of reducing downstream flows. However, it should be recognised that the volumes of runoff passing through the lakes is generally larger than for the ponds. Following the interest in how much of the available flood storage has been mobilised (see end of section 3.4) it may be noted that the total stored volume in this event was 127,000 m^3 compared with the maximum available storage of approximately 210,000 m^3 (see Appendix B 1.2). While the ornamental lakes are approximately full to their assumed maximum levels, several of the ponds still have considerable available capacity, notably Martins Heron, the Mill Pond, Oldbury, Amen Corner and Waterside Park. By contrast South Hill Park 3 seems to have exceeded its design capacity. It has not been possible within the constraints of this study to pursue these issues further.

*Table 5.11 Maximum volume (1000*m^3) stored at each pond for event 5*

Ascot Place	Saver-nake	The Warren	Martins Heron	Bay Road	Jiggs Lane	Warfield House	S.Hill Park 1	S.Hill Park 2
30.33	19.56	2.12	0.68	0.29	9.24	15.89	0.73	3.98
S.Hill Park 3	**Sports Centre**	**Mill Pond**	**Oldbury Pond**	**Amen Corner**	**Water-side Park**	**St Johns Ambulce.**	**M.Storey Car Park**	**Binfield Lake**
5.75	0.08	13.55	0.42	0.47	4.87	0.32	0.35	18.50

Tables 5.10 and 5.11 only show the impact of the ponds on local peak flows, reducing flood discharges and thus drainage requirements within the catchment. SCHEME can also be used to investigate the effect of including or excluding certain ponds on flood discharges at Binfield. For this study, the effect of excluding all the ponds specifically designed for flood control has been investigated, leaving just the three ornamental lakes. The results, based on the same fit events with the same 'best-fit' parameters, are given in Table 5.12. It should be borne in mind that these results do not include the effect of any throttling that might occur due to higher flows trying to pass through the drainage system within the catchment. These results suggest the ponds have reduced peak flows at Binfield by about 20% on average, and maybe 30% in the biggest storms. For example, event 5, with a maximum volume stored in the ponds of 127,000 m^3 (Table 5.11), and a total flood volume of 1,155,000 m^3 (Table 3.4), shows a peak flow reduction of 33%. While this is a useful reduction, it may be remembered from section 3.3.1 that despite the ponds, mean annual flood at Binfield had still appeared to increase by 56% between 1957-1973 and 1974-1990.

To investigate the opportunity for greater peak flow reduction, more detailed SCHEME runs would be required. The aim would be to determine the effectiveness of individual ponds, and the impact of alternative flood balancing strategies, such as balancing the Downmill Stream, but allowing the Bull Brook flows to pass unbalanced. Such operational issues were not within the scope of this study.

Table 5.12 The impact of flood storage in the Cut catchment

Event	Modelled Peak (m^3/s) with Ponds and Lakes	Modelled peak (m^3/s) with Lakes only	Peak flow ratio (%) with/without ponds
1	1.74	2.834	61.4
3	0.877	1.057	83.0
5	14.2	21.28	66.7
7	7.15	9.003	79.4
9	6.29	7.647	82.3
11	1.69	1.749	96.6
13	1.5	2.444	61.4
15	1.88	2.966	63.4
17	7.44	8.795	84.6
19	6.8	8.304	81.9
21	1.28	1.709	74.9
23	3.35	3.627	92.4
25	4.69	5.939	79.0
27	5.35	5.863	91.3
29	4.69	7.163	65.5
31	6.45	7.756	83.2
Mean			77.9

6. Conclusions and Recommendations

The objectives of this project were to develop and demonstrate improved flood estimation methods for mixed urban/rural catchments, with particular concern for the effects of engineering interventions such as flood storage ponds. These objectives required the establishment of a comprehensive record of rainfall and flow response throughout a study catchment, coupled with good information on the drainage system, and a verified model of flood response.

6.1 DATA GATHERING

Collecting good quality flow data from within an urban catchment presents a considerable challenge, and despite great effort in processing and quality control, the data obtained during this study retain significant uncertainties. In particular, very little good quality data have been obtained for the small rural catchments which ran dry for much of the year. Deriving stable ratings for channel sections has been complicated by seasonal weed growth and unaccountable backwater effects (possibly related to actions by riparian owners). Deriving good quality flow data for pipes where flow conditions change from sub to supercritical during storm periods has also given problems. Undoubtedly greater understanding of flow monitoring difficulties has been gained, but ideal sites for monitoring rarely exist in urban catchments. For example, the performance of flow diversion structures involving side weirs is still unknown, and any future flow monitoring associated with such structures will encounter backwater due to the effect of the downstream flow control. It is recommended that:

(1) Despite the costs involved, future studies should seek more reliable depth and velocity measurements, make greater use of depth-only monitoring at hydraulic controls and ponds, and make greater use of spot gaugings to confirm rating equations - though it is recognised that high flow periods in such catchments are generally short lived, and ratings are not necessarily 'loop free', regular and constant.

This study originally intended to produce continuous flow records to allow a greater understanding of what controls (in particular) the volume of flood response. However, given the impossibility of assembling long sequences of reliable data, the strategy was changed to the selection of isolated storm events. The aim was still to maximise the data available, so that as broad a range of hydrological conditions as possible would be covered (and replicated). A more ruthless exclusion of poorer quality data might have given greater confidence in the subsequent model studies. It is thus recommended that:

(2) Some further data correction would be necessary to extend the value of the existing data archive. Spot gauging data have not been incorporated in the derived flow ratings, and volume continuity and cross correlation checks between the various gauges could be extended.

With reference to drainage system information, collating the data for drainage structures has also given difficulties. Often storage pond and control dimensions were unknown, or their current configuration unspecified. These problems were compounded by past and continuing re-organisations of ownership and responsibility for design and maintenance. Most of the larger ponds in Bracknell were either adopted or due for adoption by Thames Water, but many smaller ponds and larger ornamental lakes remain privately owned. Unlike sewer system records, mostly held nowadays in standard STC25 format, the Water Companies have no standard archiving of flood pond data, despite some of the larger ponds falling within the scope of the Reservoirs Act 1975. It is recommended that:

(3) *Further clarification of pond details is sought from Thames Water and riparian owners, particularly where multiple outlets (which may include scour outlets) and variable sluices are included. Limited interviews with residents should also be held to determine frequency of operation.*

Concerning catchment data it is also recommended that:

(4) *The urban factors used in this study be compared with those derived for the Flood Estimation Handbook.*

Despite these concerns and recommendations, the data collected for this study represent a unique picture of the generation of storm response within a mixed urban/rural catchment, and it forms an asset of general modelling applicability.

6.2 MODEL RESULTS

Broad analysis of the data in Chapter 3 has shown the 'non-coincidence' of high flood conditions between small urban, large urban and large rural subcatchments. The dependence of flood discharges on Soil Moisture Deficit, and thus season, has also been clearly shown. Flood estimation in mixed catchments must therefore consider the responses from urban and rural areas separately, and take account of how the responses of different phase and scale combine within the wider catchment. A distributed or subcatchment approach is necessary.

The need for, and benefit of, a distributed or subcatchment approach for modelling flood behaviour in mixed urban/rural catchments is fully confirmed by the SCHEME modelling described in Chapter 5. Although other subcatchment models besides SCHEME now exist, SCHEME is the only model that is firmly based on UK data (the FSR methods) and is intended to analyse the hydrologic (rather than hydraulic) behaviour of mixed catchments. Using data for 31 flood events extracted from the full record at Binfield (16 for *fitting* and 15 for *testing*), the goodness-of-fit achieved with SCHEME has been compared with the lumped Flood Studies Report (FSR) model. At each stage of the analysis, SCHEME has consistently given better results:

- with optimum parameters, derived for each model and *fit* event individually, SCHEME gave an average correlation coefficient, R^2, of 0.95 compared with 0.91 for the FSR model.
- with overall 'best-fit' parameters, based on assessing all the *fit* events together, SCHEME gave an average R^2 of 0.94 compared with 0.90.
- with predicted parameters, based on relationships found between the 'best fit' parameters, Soil Moisture Deficit and rainfall duration, SCHEME gave an average R^2 of 0.94 compared with 0.89.
- with predicted parameters, using the same relationships, but applied to the *test* events, SCHEME gave an average R^2 of 0.94 compared with 0.84.

Similar results were also obtained based on other measures of goodness-of fit.

By treating urban and rural areas separately, SCHEME can match both the rapid urban rise time and the slower rural response, and thus reproduce the urban pulse responses superimposed on a more attenuated rural hydrograph. Predicting the volume of response remains difficult, but the good fit to observed hydrograph shape in the downstream part of the catchment, coupled with a generally good fit to data from within the urban area, give confidence that SCHEME is accurately representing the various hydrologic and hydraulic processes involved. As such, it provides a credible basis for estimating the combined effect of a network of storage ponds.

One specific result from the FSR and SCHEME modelling has been the clear inverse relationship between unit hydrograph time-to-peak (Tp) and Soil Moisture Deficit (SMD). This seems to derive from an increase in runoff from slowly responding rural areas as the catchment 'wets up', causing an extended average response time in wet conditions. The same tendency was found both in the lumped FSR modelling and in the more detailed SCHEME modelling, suggesting that even in subcatchments of fairly uniform urbanisation there are rural (unpaved) areas that contribute runoff, though more slowly, in wet conditions. Equations effectively relating Tp to SMD have been presented in this report, but these are specific to the Cut catchment.

Another result is the large departure from the standard FSR unit hydrograph shape required when modelling this mixed urban/rural catchment by a single lumped unit hydrograph. The standard QpTp=220 relationship needed to be replaced by QpTp=44.

Although this study has verified the use of SCHEME as an event model, further development of the methodology is necessary to make the model more applicable to ungauged catchments and to estimating T-year floods. Thus it is recommended:

(5) *This study has concerned just one catchment, and gives no information on how stable the parameter values are between catchments. The optimised UHTF parameter varied from 0.75 to 2.2, and UHUA was set at 1.8. It is not clear what values should be recommended for ungauged catchments (though setting all parameters to 1.0 would reproduce the FSR equations). Applications in other catchments need not necessarily be as detailed as in this study.*

(6) *An original objective of this study was to consider what design conditions should be used with SCHEME to estimate T-year floods. SCHEME is an 'event model' but the intention was to run SCHEME in a semi-continuous mode, with long observed rainfall and SMD records used to select candidate events for modelling with SCHEME, and the model peaks used in frequency analysis. This work has not been possible within the time constraints of this study, but is still needed if SCHEME (or any model) is to be used to estimate T-year floods. The various equations developed in this study to predict response parameters could form the basis for such a study based on the Cut.*

In addition to developing the general methodology, certain more detailed modelling improvements to SCHEME should be investigated:

(7) *Variation of soil type between subcatchments and its effect on proportional runoff, baseflow recharge and delay is not considered in SCHEME. Including such variability will conflict with the current manner in which baseflow parameters are derived - by analysis rather than optimisation. To include this feature, a new optimisation strategy will need to be developed.*

(8) *Some simple updates can be incorporated into SCHEME, such as the Marshall and Bayliss (1994) urban adjustment for time-to-peak, and a more hydraulic approach to modelling sewered catchments, based on the WASSP 'sewered subcatchment model'.*

(9) *SCHEME currently does not consider backwater and thus cannot model the impact of sewer surcharging in detail. Inclusion of backwater would require a significant restructuring, but would also allow SCHEME to be more easily extended as a continuous simulation model.*

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APPENDIX A. SCHEME

A.1 OVERVIEW AND USER FEATURES

SCHEME (a Sub-Catchment Hydrological Event Model for Engineers) is a 'semi-distributed' flood routing model in which storm rainfall and/or upstream hydrographs are propagated through the various sub-catchments, channels and reservoirs that make up a complex catchment. The aims are:

- to derive improved runoff estimates at the catchment outfall through better representation of runoff process within the catchment.
- to gain an appreciation of runoff response throughout the catchment and the effect of localised engineering interventions.
- to derive consistent flood hydrograph estimates at a number of 'design' locations within the catchment.

The model was originally developed for the analysis of flood runoff in partly urbanised catchments, and for the hydrological design of flood storage ponds. However it is also well suited to analysing the effect of spatial variation in topography and rainfall, assessing the impacts of chains of reservoirs, and interpolating flow hydrographs at sites within a gauging network. It is mainly concerned with hydrological response, and although it can take broad account of overbank flooding, it does not consider backwater effects in channel routing. The model has been applied to catchments ranging in size from 0.34 to 370 km^2 with a data timestep ranging from 2 to 60 minutes. Versions of the model have been also been applied to much larger catchments, without rainfall data, but with inflows from ungauged areas estimated by applying rescaling procedures to data from nearby sites.

To apply the model, the catchment is divided into a number of subcatchments on the basis of topography, land-use, and channel features. The model distributes gauged rainfall data to the subcatchments, and either determines the proportions of direct runoff, and baseflow recharge from observed hydrographs or requests 'design values' from the user. Subcatchment baseflow is routed through a linear reservoir, combined with the direct runoff, and the resulting total runoff is routed through the subcatchment using a unit hydrograph approach. Channel routing uses a linear or non-linear convection-diffusion model. On or off-line reservoirs are modelled using the normal level-pool equations combined with appropriate storage and control rules. Depending on the level of catchment information available, ranging from map data through detailed channel and flood plain data to extensive flow data, the basic models can be locally or globally tuned to improve model prediction.

The model is part parametric and part analytic, having been developed primarily to analyse rainfall-runoff response within mixed catchments. Normally, the unit hydrograph and channel routing parameters for the various subcatchments and channels are defined from 'catchment characteristics' using the procedures and equations given by the FSR (Flood Studies Report, NERC, 1975) or FSSR16 (IH,1985), but other options are also provided. The model includes seven main parameters, or global adjustment factors, which are used to re-scale the subcatchment parameters predicted from the FSR/FSSR equations. These global parameters are:

UHTF, UHSF	global factors for unit hydrograph time to peak and shape
CHCF, CHAF	global factors for channel celerity and attenuation
CRCF	the proportion of pervious area runoff taken as a constant proportion of rainfall (the remainder varies with Antecedent Precipitation Index, API)
UHUA	urban adjustment factor for unit hydrograph response time, and
IARC	runoff coefficient for impervious areas.

Two of the routing parameters (UHSF,CHAF) can usually be fixed at 1.0, leaving just two routing parameters, one loss parameter, and the urbanisation parameters to be found by optimisation. The model is fitted to individual events, and a flexible parameter optimisation procedure is provided, allowing combinations of parameters to be fixed, varied in steps, or optimised against one of five objective measures of model fit. If observed hydrograph data are not available, fixing the first six parameters at 1.0 and IARC at 0.7 will leave the subcatchment parameters to be estimated by the standard FSR/FSSR16 equations.

The model operates in four modes:

- CHECK, in which catchment data and flow data are read, to confirm their format, and then initial analyses of runoff and baseflow volumes are performed
- FIT, in which the global parameters may be optimised to improve the fit to observed rainfall-runoff data
- TEST, in which the fit of a single set of global parameters may be tested against observed rainfall-runoff data, and
- DESIGN, in which the fitted model is used to predict response from a DESIGN rainfall event, and if required, appropriate dimensions for flood storage reservoirs determined. Design rainfall may be input as a hyetograph, or specified as a combination of return-period, profile and duration as in UK Flood Studies Report (NERC, 1975) rainfall model

The model is programmed in MS FORTRAN77, version 5.1 (with MS FORTRAN graphics), and normally runs under MS-DOS. However, previous versions have been created for UNIX using UNIRAS graphics.

Following a short discussion of the model background, the following sections (A.3 to A.10) describe the basic model sequence and the rainfall, baseflow, sub-catchment, channel and reservoir modelling procedures. Section A.11 describes briefly how the model is run and how the various data files are best managed.

A.2 BACKGROUND

Distributed models of catchment runoff offer several potential advantages over simple lumped models. Variations in basin form and response between different parts of a catchment may be modelled, giving a better overall representation of catchment response. Model performance may be improved by accounting for spatial variation in rainfall and other inputs. Runoff hydrographs may be found at several points within a catchment, allowing a basin wide appreciation of runoff characteristics.

Truly distributed models may attempt these goals by solving the physical equations that represent the various catchment processes over a grid or mesh network. Such models are complex, require extensive catchment information (sometimes of a fairly esoteric nature), and their main use at present is for research. Semi-distributed models, however, claim some of the advantages of distributed models, but within a simpler framework. The catchment is treated as a number of linked subcatchments, with conceptual models used to represent subcatchment response. Such models would seem to be more immediately suited to engineering applications, and the SCHEME model described here belongs to this type.

SCHEME draws on model procedures from many sources, including the Flood Studies Report (FSR, NERC, 1975), the Wallingford Storm Sewer Package (DoE/NWC, 1981), the Australian RORB model (Laurenson and Mein, 1988), and a number of previously unpublished developments at the Institute of Hydrology. At its most basic level, the model combines the FSR unit hydrograph model (FSR Vol 1, Ch 6) with the Muskingum-Cunge river model (FSR Vol 3), a 'level-pool' reservoir routing model (developed from FSR Vol 1, Ch 7), and the UK design rainfall model (FSR Vol 2). Improvements to the FSR unit hydrograph model (FSSR5 & 16, Institute of Hydrology, 1979 & 1985) have been incorporated, and to allow the fit to be improved where rainfall-runoff observations are available, it includes the optimisation of local factors (essentially as in FSSR13, Institute of Hydrology, 1983). Besides these FSR based techniques, however, the model incorporates a new baseflow model, a revised percentage runoff model, a limited number of variations on the unit hydrograph and channel routing models, an automatic parameter optimisation procedure, and a design facility for on or off-line flood reservoirs. Observed rainfall data can be automatically distributed to the various subcatchments, and where observed hydrograph data are available within the catchment, the effect on model performance of ignoring, fitting or using the data as an upstream input can be readily assessed.

A.3 CATCHMENT REPRESENTATION AND MODEL SEQUENCE

To apply the model, the catchment is divided into a number of subcatchments on the basis of the channel network and surface characteristics. The channel network is then represented by a series of nodes placed at the subcatchment outlets, at significant confluences, reservoirs, gauging stations, and other points of interest. The area draining directly to each node, or the intervening area between pairs of nodes, is treated as a separate subcatchment, with a hydrological model for subcatchment runoff, and a hydraulic model for the flow along channel reaches between nodes. The model works with discrete storm events, starting and ending in stable baseflow conditions.

The model starts by determining the total rainfall depth and profile (distribution in time) for each subcatchment. These may be:

- read directly for each subcatchment, or more usually
- determined by SCHEME from the nearest gauge or gauges (see A.4).

At each flow gauge, observed flow data are partitioned into direct runoff and baseflow. Comparing the volume of each with the area weighted average of sub-catchment rainfall yields a percentage runoff and baseflow recharge for each flow gauge. Where another flow gauge exists upstream, the runoff and recharge figures are based only on the intervening flow and rainfall volumes.

Note that if a reservoir exists, that reservoir <u>can</u> be specified as initially drawndown below the lowest outlet. The model will predict the filling of the reservoir, but will not allow the reservoir to drain back below the lowest outlet at the end of the event. Also, the drawdown volume is not included in the runoff and baseflow calculations.

At each flow gauge, the user supplies a 'VFU' option, where the V specifies whether:

- to apply the derived runoff and baseflow parameters to the upstream subcatchments
- to ignore the hydrograph data and derive more globalised values from a hydrograph further downstream, or
- to provide alternative 'design values'.

The F option specifies whether the observed hydrograph is to be used in fitting the global parameters mentioned in section A.1, and the U option specifies whether to it is to be used to replace the modelled hydrograph as an upstream boundary condition for subsequent modelling.

Runoff calculations begin by using the subcatchment model to estimate the 'current' hydrograph at the most upstream node on the main channel. The hydrograph is then routed to the next node downstream using the channel model, the response from the intervening subcatchment is estimated using the subcatchment model and the hydrographs added to form a new 'current' hydrograph.

At confluences the current hydrograph is stored and the calculations begin again at the top of the incoming tributary branch. When modelling of that branch is complete, the stored hydrograph is added to the current hydrograph and the calculations proceed as before down the main channel.

At reservoir nodes, the usual channel and subcatchment routing is replaced with reservoir routing. On-line reservoirs are solved by the normal 'level pool' equations, while off-line ponds are solved as two or three 'coupled' reservoirs, involving flow diversion from an inlet tank and drainage to an outlet tank, or a single in/outlet tank.

At gauging stations the observed and current hydrographs may be compared and goodness-of-fit statistics calculated. The gauged hydrograph may optionally have already been used to define runoff volumes for subcatchment routing, and depending on the VFU option (described above) may now be used for parameter optimisation and/or used as the current hydrograph.

This routing process is repeated down to the final node. Areas below the final gauging station use baseflow and percentage runoff information from the final gauging station. If no flow data are available, baseflow and percentage runoff information must be supplied by the user.

A.4 DISTRIBUTING RAINFALL DEPTH AND PROFILE TO SUBCATCHMENTS

As mentioned in section A.1, the model can be run without rainfall as a simple river routing model. However, SCHEME normally needs to determine the total storm depth, RFVOL, a profile index, IPROF, and an antecedent precipitation index API for each subcatchment being modelled. The basic strategy is to estimate storm depths from daily data, and distribute depths in time according to a supplied profile. Candidate profiles may be given as either:

- hyetographs observed at a number of recording raingauges, or
- for DESIGN runs only, a single FSR design profile (eg 75% winter or 50% summer)

Where hyetograph data are given, additional data are required in order to relate daily totals to storm totals. These data are the:

- *initial* rainfall between the profile start and the previous 9 am (rainday boundary), and the
- *final* rainfall between the profile end and the next 9 am boundary

Each hyetograph is labelled by the grid reference of the raingauge. For each subcatchment, SCHEME normally uses the profile nearest its centroid, but this may be over-ridden by the user. No averaging of profiles occurs within a subcatchment in order to limit areal smoothing of peak intensity. SCHEME normalises each profile to unit volume and determines local profile factors for use in percentage runoff estimation (see section A.6).

Storm totals and antecedent precipitation data may be given as:

- point rainfall depths at a number of raingauges
- subcatchment storm depths assessed by the user in advance (from isohyets or some similar technique)
- for DESIGN runs only, a storm duration and return period (in which case the FSR rainfall model is used to derive a uniform rainfall depth).

Where point rainfall depths are supplied, each is again labelled by the grid reference of the raingauge. SCHEME will determine the weights to apply to each raingauge in evaluating subcatchment depths, based on their distance from the subcatchment centroid (SCHEME does not use digitised subcatchment boundaries). In order to limit the dominance of a raingauge very close to the centroid, SCHEME represents the subcatchment as an equilateral triangle about the same centroid but with an area 70% of the real subcatchment. Raingauge weights are determined for each vertex of the triangle based on the inverse square distance of the three nearest gauges forming a triangle around the vertex. The subcatchment weights are then taken as the average of the three vertices. The same weights are applied to the calculation of both storm totals and API.

Antecedent Precipitation data may be given as daily totals for the 5 preceding raindays, leaving SCHEME to evaluate the API_1 (see below). Alternatively, API_1 may be evaluated and input directly by the user. If storm totals are input as subcatchment storm depths, then evaluated API_1 values would also normally be given, and the *initial and final* rainfall information discussed above would be entered as null. Where point rainfall depths are given, they will usually relate to full raindays, and the ratios:

(profile depth):(profile+*initial*+*final* depth) and *initial* depth:(profile+*initial*+*final* depth)

for the subcatchment profile are used to estimate 'storm depth' and 'rainfall between 9am and the start of the event' (P_0) based on the subcatchment's weighted average daily rainfall.

API_1 is evaluated for each subcatchment as:

$$API_0 = 0.707\,P_{-1} + 0.354\,P_{-2} + 0.177\,P_{-3} + 0.088\,P_{-4} + 0.044\,P_{-5} \qquad \text{(A1)}$$

$$API_1 = API_0\,.\,0.5^{T9/24} + P_0\,.\,0.5^{T9/48} \qquad \text{(A2)}$$

Where P_{-n} is the subcatchment weighted average daily rainfall n days before the storm, and T9 is the time in hours between the start of the storm and the previous 9am.

Note also that SCHEME allows optional start and end ordinates to be specified for the supplied rainfall profiles. Any profile data outside these ordinates will be added to the *initial* and *final* depths as appropriate. Thus long storms can be split without reformatting the data, and also complete raindays of rainfall profile data may be supplied, leaving SCHEME to evaluate the *initial* and *final* depths.

A.5 BASEFLOW MODELLING

Unlike the FSR, where baseflow is treated as an adjunct to direct runoff to be added in effect as a final 'zero correction', within SCHEME baseflow is modelled as an integral part of the flow. Thus it is estimated at the subcatchment level, combined with the direct runoff and the total runoff is routed through the channel system. In any subcatchment study, a large part of the 'baseflow' observed at a downstream gauge may in fact derive from direct runoff in the upper reaches of the catchment. SCHEME therefore aims to identify subcatchment baseflow parameters from the downstream hydrograph, rather than simply separate off the baseflow component. The description below is mainly concerned with the identification of baseflow parameters while running SCHEME in CHECK or FIT mode. In TEST and DESIGN runs the parameters will be supplied directly by the user.

The baseflow model in SCHEME is based on 'contributing area' theory. Rainfall infiltration and subsurface lateral flow establish the 'wet area' of a catchment, the drainage from which forms baseflow and the surface runoff from which forms direct runoff. As baseflow and direct runoff derive from the same wet area, the hypothesis is made that they are related, and for every x% of catchment rainfall that contributes to surface runoff there is an equivalent Rx% contribution to the baseflow 'store', where R is termed the Baseflow Recharge factor. Initial losses need not be considered since they generate no surface runoff or baseflow, and indeed rainfall need not be considered since the baseflow recharge is related to surface runoff.

Neither the baseflow recharge nor its store characteristics can be determined directly 'at site'. However, the output from the baseflow store and the direct runoff pass through the same channel system, and if both the store and the channel system can be taken as linear, then the order in which the systems operate has no effect on overall response. Thus the baseflow recharge factor R and store characteristics may be derived from baseflow and runoff hydrographs observed at a downstream site, and then applied uniformly over the upstream subcatchments. The baseflow model may thus be fitted independently, rather than needing to be optimised along with the other model parameters.

At the downstream site, baseflow is only known at the start and end of an event, between which times only total flow is known. Suitable recharge factors and baseflow storage factors may however be determined by trial and error linking of flows at the start and end of the event. Thus, the baseflow store may be modelled as a linear reservoir system:

$$dS/dt = i - q, \qquad S = Kq, \qquad i = R(Q-q) \tag{A3}$$

where S is reservoir storage, t is time, i is reservoir inflow, q is reservoir outflow (*i.e.* baseflow), K is the reservoir constant, Q is total flow (thus Q-q is direct runoff), and R is the baseflow recharge factor. Combining and rearranging these equations gives:

$$dq/dt + q(1+R)/K = RQ/K \tag{A4}$$

Writing (1+R)/K as B, multiplying through by the factor e^{Bt}, and integrating gives:

$$qe^{Bt} = q_0 + (R/K).\int Qe^{Bt}dt \tag{A5}$$

Solving for a linear change in Q from Q_0 to Q_1 over a timestep Δt gives the recurrence relationship:

$$q_1 = C1\,Q_0 + C2\,Q_1 + C3\,q_0 \tag{A6}$$

where $C1 = R_0(C4 - C3)$ $\qquad C2 = R_0(1-C4)$ $\qquad C3 = e^{-B\Delta}$

$R_0 = R/(1+R)$ $\qquad C4 = (1-C3)/(B\Delta t)$ $\qquad B\Delta t = (1+R)(\Delta t/K)$

Note that the timestep, Δt, and baseflow delay, K, must be expressed in the same units.

To fit the model, first the baseflow delay is estimated from the event recession, and then, selecting suitable start and end times for direct runoff (Q_0, Q_n) and a trial recharge factor, R, successive baseflows q_i are determined until the end of direct runoff. The closing error between q_n and Q_n is used to update the trial R. Within SCHEME , the baseflow model is usually fitted when the model is first run in CHECK mode. The user specifies not a single q_n, but a region where flow has returned to baseflow, from which the model first estimates K and then iterates on R to optimise the baseflow fit to that region. The model can sometimes choose inappropriate K and R values which allow the predicted baseflow to exceed total flow during the direct runoff period. This is controlled by including such periods within the weighted 'goodness of fit' criteria. If necessary, the user can enter alternative K and R values rather than adopt the fitted values (see the discussion on concerns below).

Having determined K and R for each flow gauge, and thus identified direct runoff and baseflow, the respective volumes, or intervening volumes where an upstream gauge exists, are used to define the percentage runoff and baseflow inputs for the upstream or intervening subcatchments. Samples of the baseflow hydrographs produced by the model are shown in Fig. 5.2 of the main report.

Note that there are some concerns with the baseflow modelling approach described, though it does provide a reasonable method for assessing direct storm runoff volumes, particularly in catchments that show significant growth in baseflow between the start and end of an event. The main concern is that if during the subsequent model analyses, linearity and spatial homogeneity of runoff generation are not maintained, the parameters determined from the downstream hydrograph cannot be applied at the subcatchment level and still reproduce exactly the same downstream baseflow response, but the departures have so far proved to be small. Greater effects may occur if there are significant changes in parameter values between upstream and downstream gauges, in which cases the baseflow model may be retained, but with the parameter values defined directly by the user rather than by fitting the downstream hydrograph. Note also that lower recharge factors would be expected in urban catchments where surface runoff derives from impervious cover rather than an active 'wet area' - though urban catchments do still exhibit baseflow, derived from unpaved areas and 'infiltration' into sewer systems.

Finally, the baseflow model is not constrained by rainfall in any way, and the sum of direct runoff and baseflow recharge could exceed rainfall input. SCHEME is intended as a peak flow model, and does not attempt to maintain water balance. Significant imbalances have never been encountered, but could arise in situations where the linear baseflow model provides an inadequate match to long term recession curves. Any impact on peak flow modelling is unlikely to be great. A more likely cause of any imbalance is poor rainfall or runoff data.

A.6 RUNOFF VOLUME MODELLING

In FIT and TEST modes, overall percentage runoff is usually 'forced' to give the observed runoff volume at selected gauges. In this case, TEST mode checks just the routing parameters. However, in DESIGN mode, and optionally in TEST mode, the user may specify the baseflow and percentage runoff parameters directly. This section describes how SCHEME models percentage runoff over a storm, but it is mainly concerned with how the model parameters are defined from subcatchment rainfall depths, storm profiles, and the gauged direct and baseflow runoff volumes.

In the FSR studies, percentage runoff, PR, was allowed to vary from timestep to timestep during a storm depending on the value of Catchment Wetness Index, CWI:

$$PR_i = a.CWI_i \qquad CWI_i = 125 - SMD_i + API_i \qquad (A7)$$

where a is a parameter determined by relating direct runoff volume to total rainfall depth, SMD_i is the Soil Moisture Deficit at timestep i, starting as the daily value derived at the nearest Met. Office site, and reduced by all subsequent rainfall (unless or until it reaches its minimum value of zero), and API_i is the Antecedent Precipitation Index (see equations A1 and A2) based on a daily decay rate of 0.5, but updated at timesteps Δt (h) during the storm using a decay rate of $k=0.5^{\Delta t/24}$. Note that:

- Until SMD reaches zero, rainfall during the storm is double accounted, contributing to both SMD and API.
- Spatial variability is not be defined for SMD, but can be for API by the weighting of different raingauges. Thus applying equations A7 yields the inconsistency that different subcatchments may start with a uniform SMD, but end with different values.
- SMD calculations based on Grindley (1967) have now been superceded by the MORECS system.

For these reasons, within SCHEME, percentage runoff is given by:

$$PR_i \quad = \quad a + b\,API_i \qquad \text{(A8)}$$

where both a and b are determined by relating direct runoff volume to total rainfall depth, subject to a relationship between them defined by a model parameter CRCF (see below). Note that API_i is in effect the status of an upper soil moisture store that is replenished by rainfall, but drains by the decay factor k over each successive timestep.

Considering a single subcatchment of area A, and writing the API decay constant for a single timestep as k, the volumes of runoff V generated by rainfall P in the first few timesteps of the event are given by:

$$V_1 = A\,.P_1\,.PR_1 = A\,.P_1\,(a + b\,API_1)$$

$$V_2 = A\,.P_2\,.PR_2 = A\,.P_2\,(a + b\,API_2) = A\,.P_2\,(a + b\,k\ \ API_1 + b\,k^{½}\,P_1)$$

$$V_3 = A\,.P_3\,.PR_3 = A\,.P_3\,(a + b\,API_3) = A\,.P_3\,(a + b\,k^2\ API_1 + b\,k\,k^{½}\,P_1 + b\,k^{½}\,P_2)$$

$$\text{or}\quad V_i = \qquad A\,.P_i\,(a + b\,k^{i-1}\,API_1 + b\,k\,SPI_i) \qquad \text{(A9)}$$

where SPI_n ($= k\,SPI_{n-1} + k^{½}\,P_{n-1}$) is a Storm Precipitation Index with an initial value of $SPI_1 = 0$. Summing over the whole storm duration gives:

$$\sum V_i = A\,(\,a.\sum P_i + b.API_1\,\sum(k^{i-1}\,P_i\,) + b.\sum(SPI_i\,P_i\,)\,)$$

Writing p_i and spi_i for values derived from the normalised rainfall profile (*i.e.* from dividing by the total subcatchment rainfall depth $RFVOL=\sum P_i$) gives

$$\sum V_i = A\ (\,a\,.RFVOL + b.API_1\,.RFVOL.\,\sum(k^{i-1}p_i\,) + b.RFVOL^2\,.\,\sum(SPI_i\,p_i)\,)$$

$$\text{or}\quad \sum V_i = A\ (\,a\,.RFVOL + b.API_1\,.RFVOL.GPROF\ \ +\ b.\,RFVOL^2\,.HPROF) \qquad \text{(A10)}$$

Where GPROF ($=\sum k^{i-1}\,p_i$) and HPROF ($=\sum SPI_i\,p_i$) are functions of the normalised profiles, and may be evaluated just once when the profile data are first read (see section A.4).

Applying equation A10 now to just the pervious area, AP, of the catchment, incorporating a fixed runoff coefficient (IARC) from the impervious area, AI, and summing over all subcatchments upstream of the flow gauge gives the total surface runoff, GVOL, as:

$$GVOL = IARC.\Sigma\{AI_j.RFVOL_j\} + a\,\Sigma\{AP_j.RFVOL_j\} + b\,(\Sigma\{AP_j.RFVOL_j.API_j.GPROF_j\} + \Sigma\{AP_j.RFVOL_j^2.HPROF_j\}) \qquad (A11)$$

where the suffix j gives the subcatchment number, and the API is now taken as the value at the start of the storm.

It may be noted in this equation that the profile factors are defined from one of the recording raingauge profiles, but the RFVOL and API values are weighted averages formed from the nearest 'daily' raingauges.

In equation A11, IARC is one of SCHEME's global model parameters, but *a* and *b* are not. SCHEME has been parameterised to explore the question 'How variable is the pervious area percentage runoff over a storm?'. Thus writing QTOT for total pervious area runoff, defined as the last two terms in equation A11, the global parameter CRCF adopted for SCHEME is given as:

$$CRCF = (a\,\Sigma\{AP_j.RFVOL_j\}) / QTOT \qquad (A12)$$

Equations A11 and A12 are then used to define *a* and *b* for each flow gauge. A CRCF value of 1.0 means b=0, and a CRCF value of 0.0 means a=0.

Note, however, that if the gauged surface runoff is less than the first term of equation A11 then SCHEME sets *a* and *b* to zero, and reduces IARC below the input value until the correct volume is obtained. Also, if the pervious area runoff coefficient exceeds IARC, then SCHEME sets *b* to zero, sets *a* equal to IARC, and increases them both until the correct volume is achieved.

Note also, that percentage runoff as derived from equation A8 is not constrained to be less then 100%. However, values above 100% have rarely been estimated, and in each case could be related to data inadequacies.

The parameters *a* and *b* derived from observed hydrographs are applied to all subcatchments upstream, or until another 'active' gauging station is encountered. As described in section A.3, the user may elect to exclude certain flow data from the calculations using the 'VFU' flags. If the response from any subcatchment does not pass an 'active' gauging station, its baseflow and runoff parameters would be defined from the final gauging station. If no 'active' gauging station is defined, runoff volume data must be supplied by the user.

Note that development of the runoff volume model to allow *a* and *b* to vary within a gauged area based on, for example, soil type would be possible. However, as baseflow is currently linked to direct runoff, areas yielding greater direct runoff would also yield higher baseflow. To correct this inconsistency would require a new approach to the estimation of baseflow parameters based on global optimisation.

A.7 SUBCATCHMENT ROUTING

Subcatchment runoff modelling in SCHEME is based on a unit hydrograph approach. Effective rainfall is determined from equations A9 using the parameters *a* and *b* obtained at the next active gauging station downstream. The Baseflow contribution is derived from equations A6 using parameters R and K obtained for the same station. Baseflow and effective rainfall are then combined and convolved with the unit hydrograph. The resulting subcatchment response is added directly to the hydrograph at the downstream model node (see section A.3), and lateral inflow to channels between nodes is not explicitly modelled.

For each subcatchment, a number of options are available for determining the unit hydrograph shape and time-to-peak (or lag-time). These include:

- a standard parametric shape (*e.g.* FSR triangle, CIRIA, Nash cascade) with time scaling parameter (time to peak or lag time) determined by SCHEME from an equation based on catchment characteristics.
- the standard parametric shape, but with shape parameter (*e.g.* the QpTp of an FSR triangle, or the Nash 'n') and time scaling parameter directly specified (fixed) for the subcatchment.
- a subcatchment specific unit hydrograph given as a sequence of ordinates.

The second two options are provided for *specific* subcatchments where flow is gauged and a local estimate of time to peak or unit hydrograph can be derived. This will rarely be the case, and normally the first *default* option will be used. Within any one run, the same *default* parametric shape and time scale equation will be used for all subcatchments. The normal choice would be the FSR triangular unit hydrograph, with time to peak defined by the FSSR16 equation (see below). However the same Tp equation may be used with a smoother Nash cascade shape (Nash, 1960), or else the now obsolete CIRIA hydrograph shape and lag time equation (from an earlier edition of Hall *et al*, 1993). Shape and time parameters may be varied globally across (only) those subcatchments using the *default* option by changing one or more of three global model parameters: UHUA, UHTF, and UHSF.

In FSSR16 (IH, 1985), the time to peak, Tp(0) (h) of the instantaneous unit hydrograph is given as:

$$Tp(0) = 283\ S1085^{-0.33}\ (1+URBAN)^{-2.2}\ SAAR^{-0.54}\ MSL^{0.23} \tag{A13}$$

where S1085 is mainstream slope (m/km), from 10% to 85% of stream length from outlet
URBAN is the fraction of catchment urbanised
SAAR is catchment average annual rainfall (mm)
MSL is mainstream length (km)

In FSSR16, this equation is applied only to complete catchments, but in SCHEME it is used to estimate the response time of 'downstream' subcatchments where the mainstream passes into another subcatchment upstream. In such cases S1085 and MSL are replaced by the subcatchment length, L, and slope, S, starting in the mainstream but then following up the largest 'side tributary' that remains wholly within the subcatchment. The length, slope, and urban fraction must be evaluated for each subcatchment, but a single, catchment average SAAR value is used for all subcatchments. It may be noted that SCHEME also requires the subcatchment areas and impervious fractions (see equation A11), but impervious fraction may be estimated as GIMP*URBAN, where GIMP is the global impervious factor for URBAN area, given as 0.3 in FSSR5.

For each subcatchment (suffix j), using equation A13, SCHEME first derives a rural estimate of time to peak, $Tp_r(0)_j$. This estimate is then adjusted globally during model calibration by varying the model parameters UHTF and UHUA as

$$Tp(0)_j' = UHTF\ .\ Tp_r(0)_j\ .\ (1+URBAN_j)^{-2.2*UHUA} \tag{A14}$$

and finally the Tp(0)' are converted to Tp(T)' for the required data interval, T (or Δt in section A.5):

$$Tp(T)_j' = Tp(0)_j' + T/2 \tag{A15}$$

The peak discharge (m^3/s) of the triangular unit hydrograph is then defined by

$$Qp_j = UHSF \,.\, 2.2 \,.\, A_j \,/\, Tp(T)_j' \qquad \text{(A16)}$$

where UHSF is a global model parameter adjusted during optimisation
A_j is the subcatchment area (km^2), and
$Tp(T)_j'$ is the subcatchment time to peak (h) of the T hour unit hydrograph

Equation A16 is essentially the well-known FSR equation QpTp=220, where Qp is expressed in $m^3s^{-1}/100\ km^2$. Unit volume considerations require QpTb=555, where Tb is the timebase (h) of the triangular unit hydrograph. Thus the standard FSR shape has a slight skew, with Tp approximately 40% of Tb; UHTF allows this skewness to be adjusted globally.

Note from equations A14 and A16 that putting UHTF, UHSF and UHUA all equal to 1.0 reduces the model to the standard FSR equations. If the Nash cascade unit hydrograph shape is adopted, the model parameter UHSF defines the number of reservoirs in the cascade.

The basic catchment characteristics used to define the subcatchment routing model may be summarised as: catchment average annual rainfall (SAAR, mm), subcatchment area, length, and slope (A, km^2, L, km, S, m/km), and urban fraction (URBAN). The values for each subcatchment are held in the catchment data file (see section A.11), together with a flag to identify those *specific* subcatchments where a locally derived time to peak or unit hydrograph is to be used in preference to the *default* parametric form.

A.8 CHANNEL ROUTING

As described in section A.3, the channel routing model is used to convey hydrographs from upstream to downstream nodes along the channel network. In SCHEME, channel routing is based on the convective-diffusion equation,

$$\partial Q/\partial t \;+\; c\,\partial Q/\partial x \;=\; a\,\partial^2 Q/\partial x^2 \qquad \text{(A17)}$$

with the wave speed, c, and attenuation parameter, a, taken as constant or as functions of discharge, Q. Both constant and variable wave speed models are normally solved by the Muskingum-Cunge finite difference scheme (see FSR Vol 3, NERC, 1975; Price, 1985).

$$Q_{D1} = C1.Q_{U0} \;+\; C2.Q_{U1} \;+\; C3.Q_{D0} \qquad \text{(A18)}$$

where $C1 = \{\ \Delta t + 2\Delta x/(c_{U0}+c_{U1}) - (b_{U0}+b_{D0})\ \}/C0$, $\quad C2 = \{\ \Delta t - 2\Delta x/(c_{U0}+c_{U1}) + (b_{U1}+b_{D1})\ \}/C0$,

$C0 = \{\ \Delta t + 2\Delta x/(c_{D0}+c_{D1}) + (b_{U1}+b_{D1})\ \}$, $\quad C3 = 1 - C1 - C2 \quad$ and $\quad b=a/c^2$

where the suffices U, D, 0, and 1 refer respectively to values at the Up and Downstream ends of the space step, Δx, and the start and end of the timestep, Δt. The space step is taken as:

$$0.5\,(c_{reference}\Delta t) \;<\; \Delta x \;<\; 1.6\,(c_{reference}\,\Delta t) \qquad \text{(A19)}$$

Note that the right hand side of equation A18 involves c and a values estimated at the Q_{D1} evaluated on the left hand side, and therefore involves an iterative solution. Note also that the dependence of space step on a reference wave speed can lead to discontinuities in the 'goodness of fit', causing difficulties with parameter optimisation. For this reason, an alternative method for the constant parameter case has been included, based on the channel impulse response (see Nes and Hendriks, 1971). However, as Price (1985) has shown, the methods converge on different solutions and are not directly comparable.

Equation A17 does not account for backwater effects in either constant or variable parameter form, but the variable form does allow channel lag to change with discharge, and can model the effect of flood plain storage. However, these features are included in SCHEME only so that they can be modelled where data show they have an effect; SCHEME was not intended as a hydraulic routing model, and the need for anything other than approximate channel cross-sections was not envisaged.

In applying equations A17/A18, the parameters c and a for each channel reach may be taken as:

- constant: based on an estimate of flood travel time between the upstream and downstream nodes, and on channel length, breadth and slope
- variable: based on Manning's equation assuming a wide rectangular channel
- variable: based on Manning's equation, but including a simplified flood plain geometry, or
- variable: read in directly as functions of Q.

One of the first two options is chosen as the *default* estimation method, for which global optimisation of the c and a parameters is available (see below). The last two options would be used in *specific* reaches, normally where there is better individual information available on the c and a relationships. Consequently, in these *specific* reaches, the estimated parameters are not adjusted by optimisation.

In most cases, it is expected that the constant parameter Muskingum-Cunge model will be chosen as the *default* method, and SCHEME includes several methods for estimating wave travel time:

- from the difference in $Tp_r(0)$ evaluated at the upstream and downstream nodes (using equation A13 with URBAN=0),
- from an area weighted difference in $Tp_r(0)$ between upstream and downstream nodes,
- from an area weighted difference in $Tp_r(0)$, with additional weighting at confluences;
  from the Manning 'kernel' for the channel reach, $CHANL/CHANS^{0.5}$, where CHANL and CHANS are the channel length (km) and slope (m/km) between nodes,
- from an 'area adjusted kernel' allowing for growth in channel capacity with total upstream catchment area, $CHANL/(CHANS^{0.4} TOTA^{0.2})$,
- from a Manning equation based on channel slope, breadth, roughness, and upstream peak flow.

In all but this last case, travel time, ΔT (h), may be adjusted for urbanisation/channelisation based on either Manning's n, or on the factor $(1+URBAN)^{-2.2}$ from equation A13 (where URBAN is taken from the subcatchment that contains the channel reach). Wave speed c (m/s) is then estimated as:

$$c_j = (1000.CHANL) / (3600.\Delta T) \qquad (A20)$$

For the Manning equation, wave speed, c (m/s), is estimated directly, assuming a wide rectangular channel, as:

$$c_j = dQ/dA = (5/3)\,(\,(0.001.CHANS)^{0.3} .CHANB^{-0.4} .n^{-0.6}\,)\,Q^{0.4} \qquad (A21)$$

where CHANS and CHANB are channel slope (m/km) and breadth (m), n is Manning's n, and Q is taken as the upstream peak flow (m^3/s).

In all cases, the attenuation parameter is estimated as:

$$a_j = Q / (0.002 .CHANS .CHANB) \qquad (A22)$$

The c and a parameters are then optimised as

$$c_j' = \mathrm{CHCF}.c_j \qquad \text{and} \qquad a_j' = \mathrm{CHAF}.a_j \tag{A23}$$

where CHCF and CHAF are the SCHEME global model parameters to adjust wave speed and attenuation across all subcatchments.

If the variable parameter Muskingum-Cunge method is chosen as the *default* method for estimating c and a, then equations A21 to A23 are still used, but with Q taken as the instantaneous flow, upstream or downstream as appropriate (see equations A18).

For those *specific* reaches where the variable parameter method including simplified flood plain geometry has been selected, inbank flows are represented by a rectangular channel (equation A21 and A22), and the flood plain flow by a shallow 'v' section, for which the Manning equation gives a wave speed discharge relationship of:

$$c_j = (2^{1.75}/3)\,(\,(1000.\mathrm{CHANS}^{0.375}\,X^{-0.25}\,n^{-0.75})\,Q^{0.25} \tag{A24}$$

where X is the ratio of channel width to depth. Transition between equations A21 and A24 is based on a cubic spline fitted between reference Q values of one and two times a 'bankfull' discharge supplied by the user. The attenuation parameter a_j is given by the equation A22 as before, but with CHANB determined from the flow area derived for twice the input bankfull discharge.

The basic catchment characteristics used to define the channel routing model may be summarised as: channel length, slope, breadth and roughness (CHANL, km, CHANS, m/km, CHANB, m, and Manning's n). The values for each channel, together with a flag to identify any *specific* reaches where more individual modelling is required, are held with the subcatchment data in the catchment data file (see section A.11).

A.9 RESERVOIR ROUTING

Reservoir routing is based on the usual 'level pool' equations, expressing the change in reservoir storage S from timestep 1 to 2 on the assumption that inflow, I, and outflow, Q, vary linearly over the time interval Δt:

$$(S_2 - S_1)/\Delta t \;=\; (I_1 + I_2)/2 \;-\; (Q_1 + Q_2)/2, \qquad \text{or rearranging}$$

$$2\,S_2/\Delta t + Q_2 \;=\; 2\,S_1/\Delta t \;- Q_1 \;+\; (I_1 + I_2) \tag{A25}$$

where both the storage and outflow are functions of the level (or head) in the reservoir. Currently the inflow does not consider the effect of rain falling directly onto the reservoir surface. Noting that at the start of a timestep all the terms on the right hand side of equation A25 are known, a solution is found by Newton-Raphson iteration:

$$h_b = h_a - f\{h_a\}/f'\{h_a\} \tag{A26}$$

where $f(h_a)$ is the error obtained from equation A25 using an initial head estimate, h_a, and $f'(h_a)$ is the differential with respect to h of the left hand side of the equation, evaluated at h_a. In SCHEME the iteration is continued until the head error is less than 0.1 mm; then the outflow Q is estimated at mid interval to assess the linearity assumption. If the departure exceeds 1% the timestep is halved, equation A25 is solved, and the linearity checked again, until the criterion is met.

In SCHEME, the storage-head relationship is defined from an area-head relationship given as:

- a power-law equation of the form A= A0 + AGROW.(head-HZERO)^{XGROW}, or
- a table of head and area data points.

Similarly, as described further below, the outflow-head relationship is given by:

- a number of power-law equations of the form Q= C.(head-H0)^{XC} for different controls and ranges of control, or
- the dimensions of certain 'standard controls' having different control ranges (e.g. culverts with part-full and full-bore ratings), or
- a table of head and outflow data points.

For on-line reservoirs, where all the upstream flow passes through the reservoir, the number of controls and ranges is virtually unlimited. However, for off-line reservoirs, where reservoir inflow comes from flow diversion at an inlet structure (*e.g.* a throttle pipe and side-weir), the number of controls is limited and the order in which they are presented in the data files is fixed. Off-line reservoirs are modelled as two or three 'coupled' reservoirs, depending on their outlet structures, with free, drowned, or one-way flows between the reservoirs determined as described below (see also Fig. A1).

For off-line reservoirs with combined inlet and outlet structures, two reservoirs are involved:

- Reservoir 1 is the inlet/outlet structure, with
  - storage $X\{f\}$ a function of head, f,
  - inflow I from upstream hydrograph, I,
    R from control 3,
  - outflow C from control 1, the 'carry-on'/controlled 'outflow', and
    D from control 2, the 'diversion' into the off-line reservoir.
- Reservoir 2 is the off-line reservoir itself, with
  - storage $Y\{g\}$, a function of head, g;
  - inflow D from control 2; and
  - outflow R from control 3, the return control, and
    E from controls 5 etc, emergency and any uncontrolled flows from the reservoir.

Note that: controls 2 and 3 may be drowned by downstream conditions,
control 2 would allow reverse flow, but
control 3 is assumed to be non-return (flap valve)
control 4 must be included as a null control

Inserting these storages and flows into the storage equations for the two reservoir system gives:

$$
\begin{aligned}
2\,X\{f_2\}/\Delta t + C\{f_2\} + D\{f_2, g_2\} - R\{g_2, f_2\} &= 2\,X\{f_1\}/\Delta t - C\{f_1\} - D\{f_1, g_1\} + R\{g_1, f_1\} + I_1 + I_2 \\
2\,Y\{g_2\}/\Delta t + R\{g_2, f_2\} + E\{g_2\} - D\{f_2, g_2\} &= 2\,Y\{g_1\}/\Delta t - R\{g_1, f_1\} - E\{g_1\} + D\{f_1, g_1\} \\
Q_2 &= E\{g_2\} + C\{f_2\} \qquad \text{(A27)}
\end{aligned}
$$

These equations are solved simultaneously using a multi-dimensional Newton-Raphson method.

Reservoir Schematic Diagram

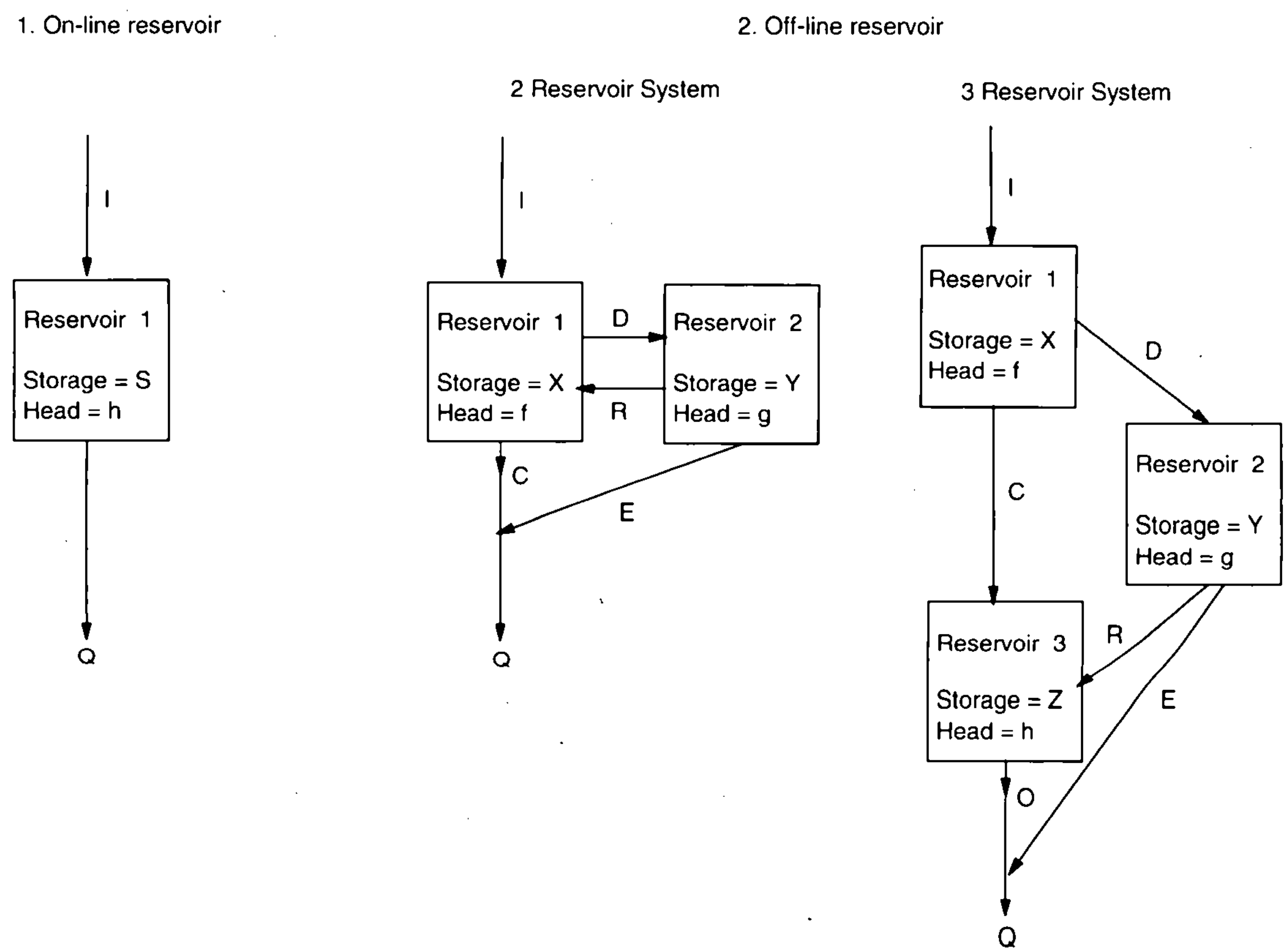


Figure A1

For off-line reservoirs with separate inlet and outlet structures, three reservoirs are involved:

- Reservoir 1 is the inlet structure, with
  storage $X\{f\}$ a function of head, f,
  inflow I from upstream hydrograph, I,
  outflow: C from control 1, the 'carry on' or 'bypass' flow, and
  D from control 2, the 'diversion' into the off-line reservoir;
- Reservoir 2 is the off-line reservoir itself, with
  storage $Y\{g\}$, a function of head, g,
  inflow D from control 2,
  outflow R from control 3, the return control, and
  E from controls 5 etc, emergency and any uncontrolled flows from the reservoir;
- Reservoir 3 is the outlet control, with:
  storage $Z\{h\}$, a function of head, h,
  inflow C from control 1,
  R from control 3, and
  outflow O from control 4, the final controlled outflow.

Note that: controls 1, 2 and 3 may be drowned by downstream conditions,
controls 1 and 2 would allow reverse flow, but
control 3 is assumed to be non-return (flap valve)
any channel lag between Reservoir 1 and 3 is ignored.

Inserting these storages and flows into the storage equations for the three reservoir system gives:

$$
\begin{aligned}
2\,X\{f_2\}/\Delta t + C\{f_2, h_2\} + D\{f_2, g_2\} &= 2\,X\{f_1\}/\Delta t - C\{f_1, h_1\} - D\{f_1, g_1\} + I_1 + I_2 \\
2\,Y\{g_2\}/\Delta t + R\{g_2, h_2\} + E\{g_2\} - D\{f_2, g_2\} &= 2\,Y\{g_1\}/\Delta t - R\{g_1, h_1\} - E\{g_1\} + D\{f_1, g_1\} \\
2\,Z\{h_2\}/\Delta t + O\{h_2\} - C\{f_2, h_2\} - R\{g_2, h_2\} &= 2\,Z\{h_1\}/\Delta t - O\{h_1\} + C\{f_1, h_1\} + R\{g_1, h_1\}. \\
Q_2 &= E\{g_2\} + O\{h_2\} \qquad \text{(A28)}
\end{aligned}
$$

As before, these equations are solved simultaneously using a multi-dimensional Newton-Raphson method.

The same options for determining reservoir area-head relationships and discharge-head relationships exist for off-line reservoirs as for on-line reservoirs. In both cases, the area and outflow relationships will normally be defined in advance, and the data will be held with the subcatchment and channel data in the catchment data file. However, if the 'power-law' forms of relationships are used, the various coefficients *etc* can be set as undefined, allowing them to be adjusted directly by the user at 'run time'. This allows reservoir controls to be designed and optimised.

A.9.1 The outflow-head equation

The outflow-head relationships available in SCHEME are very flexible. The basic equation is:

$$Q = f.C.(h-H0)^{XC} \qquad \text{for } h < HL \qquad \text{(A29)}$$

where Q is outflow (m^3/s) h is head (m)
C is the structure coefficient, H0 is the zero level (m),
XC is the structure exponent, HL is given as the upper limit of applicability (m)
and f is a drowned flow correction (see below)

This basic form allows several options:

- Variable coefficient: if C is given as zero, a table of h:C values is entered to define how C varies with h. If XC is zero, the h:C become h:Q values (but h:Q values can be input directly)
- Multiple controls: with equations repeated in sequence, if HL is zero, the current equation has no upper limit, and the following equation represents a control in parallel to the current equation
- Multiple ranges: with equations again given in sequence, if HL is given a non-zero value greater than H0, the current equation switches to the following equation at h=HL.
- Adjust at run-time: if any value is specified as -1, its value will be read/adjusted at run time.

The multiple ranges option is extended by the use of an additional 'flag' to denote a 'standard control'. In this case, the dimensions of the control are read, and the SCHEME determines the corresponding C, H0, XC and HL values within effective ranges. Thus:

- Flag 'W', a weir: only C and H0 are read; SCHEME sets XC=1.5 and HL to 0.
- Flag 'O', on drowned orifice/sluice (a special case that must follow a weir): only C and H0 are read; SCHEME sets XC=0.5 and HL to 0, and redefines HL for the weir to the level at which its discharge equals that of this orifice (i.e. where the weir drowns).
- Flag 'B', a box culvert/sluice: reads width B (m), zero level, H0, and height, D (m); SCHEME defines a two part compound rating (see below)
- Flag 'C', a circular culvert/standard orifice: reads diameter, D (m), zero level H0, and number, N, of bores; SCHEME defines a three part compound rating

Culverts are modelled assuming they operate under inlet control, using equations developed from those presented by Henderson (1966). A box culvert of width B and height D is modelled using two equations:

$$Q = 1.56\, B\, (h - H0)^{1.5} \qquad h - H0 < 1.2D \qquad \text{(A30)}$$
$$Q = 2.65\, B\, D\, (h - H0 - 0.6D)^{0.5} \qquad h - H0 > 1.2D \qquad \text{(A31)}$$

and N parallel circular culverts of diameter D are modelled using three equations

$$Q = 1.35\, N\, D^{0.6} (h - H0)^{1.9} \qquad h - H0 < 0.8\,D \qquad \text{(A32)}$$
$$Q = 1.24\, N\, D\, (h - H0)^{1.5} \qquad 0.8\,D < h - H0 < 1.2\,D \qquad \text{(A33)}$$
$$Q = 2.10\, N\, D^2 (h - H0 - 0.6D)^{0.5} \qquad h - H0 > 1.2\,D \qquad \text{(A34)}$$

Finally, the offline pond controls, C, D and R, are allowed to drown and C and D are allowed to produce reverse flow. For all control forms, drowned flow correction is made using the Crump weir formula which allows for a modular limit, $(h_2\text{-}H0)/(h_1\text{-}H0)$ of 0.75 between the upstream and downstream heads, h_1 and h_2 respectively. The Crump equation is:

$$f = \left(1 - (4(h_2-H0)/(h_1-H0)-3)^{1.5} \right)^{0.45} \qquad (h_2-H0) > 0.75\,(h1-H0) \qquad \text{(A35)}$$

The Villemonte formula ($f=(1-((h_2-H0)/(h_1-H0))^{1.5})^{0.385}$) for a thin plate weir is also included in SCHEME but it reduces flows as soon as the downstream level reaches the weir sill, which seems too soon. Further research to improve the modelling of drowned flow for a range of controls would be beneficial.

A.10 OPTIMISATION

Within SCHEME, the seven global parameters, UHTF, UHSF, CHCF, CHAF, CRCF, UHUA and IARC, may be fitted by optimisation. SCHEME includes a flexible parameter optimisation procedure in which selected parameters can be fixed, changed in steps over a range, and optimised freely within a range. The optimisation procedure is based on the Rosenbrock (1960) rotating orthogonal co-ordinates algorithm.

Currently, storm events must be optimised individually. Combined optimisation of several storms was available in an earlier version of SCHEME, but this feature has not yet been re-introduced following an upgrade of the modelling structure.

A range of objective measures of fit may be optimised

- Peak error $(Qp_{estimated} - Qp_{observed})/Qp_{observed}$
- Av.Abs.error $(\Sigma\,|Q_{estimated} - Q_{observed}|\,)/\Sigma Q_{observed}$
- R.M.S.error $(N\,\Sigma(Q_{estimated} - Q_{observed})^2)^{½} / \Sigma Q_{observed}$
- W.R.M.S.error $(N\,\Sigma\,|Q^2_{estimated} - Q^2_{observed}|)^{½} / \Sigma Q_{observed}$
- T.R.M.S.error $((1/N')\Sigma(Q_{estimated} - Q_{observed})^2)^{½}/((1/N)\Sigma Q_{observed})$ while $Q_{observed} > 0.5\,Qp_{observed}$
- R.M.S % error $((1/N)\,\Sigma(1 - Q_{estimated}/Q_{observed})^2)^{½}$

Where Qp is the peak of the hydrograph, and Q are the individual ordinates. All summations are over the full N ordinates available for the event, except in the truncated T.R.M.S. error, where the first summation is over just the N' ordinates for which $Q_{observed} > 0.5\ Qp_{observed}$. In general the full RMS error and the TRMS error have proved most 'reliable' for optimising peak flow performance.

Remembering the 'VFU' flag discussed in section A.3, these statistics are determined separately for each location where the 'F' flag is set, and the results combined. By expressing errors in proportion to mean or peak observed flows at the site, these statistics prevent the larger gauged catchments, that generally yield larger numerical errors, from dominating the overall error statistics.

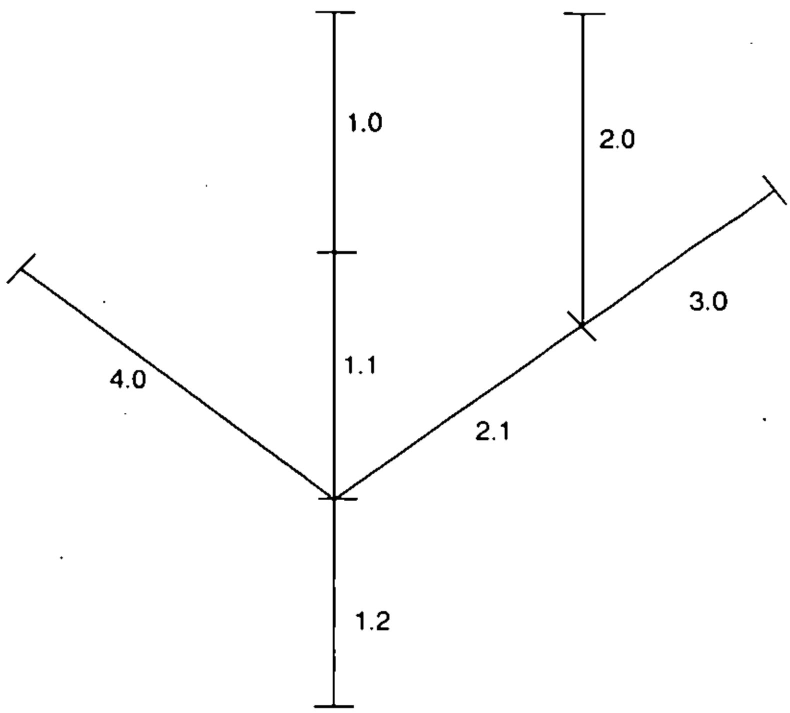
A.11 MODEL IMPLEMENTATION

A.11.1 Node labels

As described in section A.3, the catchment is divided into subcatchments and represented by a series of nodes placed at the subcatchment outlets, confluences, reservoirs, gauging stations, and other points of interest. The sequence of calculations by which flow is generated from rainfall and propagated from node to node is controlled by an ordered list of '*branch.reach*' codes, similar to those used in the WASSP and WALLRUS sewer models. These codes serve as labels for the nodes, the upstream inter-nodal subcatchments and the channels. Within SCHEME they are also used to reference gauging station nodes and locations where hydrograph output is required.

Figure A.2 gives a nodal representation of a simple example catchment, together with the *branch.reach* codes used to sequence the modelling. The codes start with the mainstream (normally *branch* 1) and work from upstream to downstream (normally incrementing *reach* from an initial value 0). At confluences the current *branch* is suspended while the tributary nodes are traversed (also upstream to downstream) using a new (incremented) *branch* and (re-initialised) *reach*. This procedure is nested as necessary for any sub-branches. There is no limit on the number of branches that can meet at a confluence, but the maximum number of confluences that may be nested within a branch is currently fixed at four. At the next node below a confluence, the *branch* code returns to

Nodal representation of sample catchment



Calculation sequence

Branch reach	MODEL OPERATION	
1.0	SUBCAT 1.0	
1.1	CHAN 1.1,	ADD SUBCAT 1.1
2.0	STORE1.1,	SUBCAT 2.0
3.0	STORE 2.0,	SUBCAT 3.0
2.1	ADD STORED 2.0, CHAN2.1, ADD SUBCAT 2.1	
4.0	STORE 2.1,	SUBCAT 4.0
1.2	ADD STORED 2.1, ADD STORED 1.1, CHAN 1.2, ADD SUBCAT 1.2	

Where
SUBCAT = Model subcatchment hydrograph
CHAN = Route hydrograph along channel

Figure A2

that of the first (mainstream) *branch* to enter the confluence, and the *reach* code is incremented with respect to that *branch*. A node is not required at the top of the outgoing reach from a confluence unless the combined hydrograph at the confluence is specifically required, in which case a special junction node is provided.

In addition to a *branch.reach* code, each node has up to three 'flags' to indicate special features. The first flag indicates reservoir or junction nodes (involving no subcatchment or channel routing). The second and third flags invoke specific options for subcatchment and channel routing.

The example *branch* and *reach* codes shown on Fig. A.2 are (for simplicity) single digits, but each may extend to 3 digits. Current program limits allow a maximum of 100 nodes, 80 subcatchments and 4 nested confluences. In practice, having *branch* and *reach* codes increase sequentially is convenient for the user, but not necessary to the program. *Branch* codes can take any value not already active (or stored), while *reach* codes need only be distinct within the *branch*. This allows the subcatchment configuration to be changed without the need to redefine all the *branch.reach* codes.

A.11.2 File management and model operation

All the subcatchment, channel and reservoir data needed to define the generation and routing of flow through the catchment (e.g. areas, soils, URBAN fractions, channel lengths, slopes, reservoir areas, controls) are stored against the ordered *branch.reach* codes in a catchment data file. Rainfall and flow data are stored in storm data files. The model accesses the files and requests further information directly from the user, in particular: the level of output required; the nodes at which hydrograph output is required; the nodes at which hydrographs should be used for fitting; the type of model run required; the parameters to fix or to optimise and their respective ranges. Hydrographs may be plotted, saved to file, or plotted with other previously saved hydrographs for comparison. At the end of a model run, the program offers a choice to change run type, parameters, storm, or catchment.

The catchment and storm data files for each catchment are best held in separate directories. The user may either change to the required directory before running SCHEME, or tell SCHEME the PATH for the data files. The catchment data file is best given the file extension CAT. Different versions of the file (corresponding perhaps to different urbanisation levels) may have other extensions, but it may be more convenient to use different names (e.g. RURAL.CAT, URBAN.CAT). Storm data files are best given the same name as the catchment data file, but with different extensions. Output hydrograph filenames generated by the model use the first character of the catchment data filename followed by the node label ('#' replacing '.') with the file extension taken from the storm data file.

Full details of how to run SCHEME, including the formats of the various data files area not included here, but are available in a separate users manual.

APPENDIX B. Data archive

B.1 CATCHMENT DATA

To analyse the flood response of the Cut at Binfield using the SCHEME model (see Appendix A), the catchment must be divided into a number of subcatchments based on land form, land use, and channel features such as confluences, reservoirs, gauging stations and other points of interest. The land use and major drainage features of the Cut at Binfield are shown on Fig. B1, with (in white) the catchment boundaries to the principal flow and level recorders used in this study. The companion Fig. B2 shows the same catchment boundaries together with (a) the location of the 18 (numbered) storage ponds and lakes modelled in this study, (b) intermediate catchment boundaries (in black) to the storage ponds and main confluences, and (c) the *branch.reach* codes (in red) used in SCHEME to label the subcatchments and define how the flow converges to the catchment outlet (see Appendix A). In all, the catchment has been divided into 42 subcatchments. This section describes how the subcatchment boundaries were defined and how the 'catchment characteristics' of each subcatchment have been determined.

The SCHEME model uses the standard 'catchment characteristics' defined in the Flood Studies Report (NERC, 1975, see example in Vol. 1 p458-465). These are:

- the catchment area, A (km^2)
- the length, L (km), and slope, S (m/km), of the main drainage channel
- the proportion of the catchment urbanised, URBAN
- a measure of climate (Standard Average Annual Rainfall, SAAR mm),
- the soil type SOIL (weighted average number),
- and measures of pre-storm catchment 'wetness', API (mm) and SMD (mm).

Modelling of the various flood storage ponds and lakes was based on the control dimensions and storage curves supplied by Thames Region of the Environment Agency (EA, formerly the National Rivers Authority), or Bracknell Forest Borough Council (BFBC), or was determined from maps and plans.

However, several significant differences exist between this study and the FSR example:

- The catchment has been divided into subcatchments draining to 'nodes' at selected points on the channel network. Area, A, length, L, slope, S, and URBAN have been derived for the subcatchment local to each node (i.e. not draining via an upstream node). Where an upstream node exists, the 'main stream' between the downstream and upstream node was used to define the length, CHANL, and slope, CHANS, used for channel routing, but subcatchment length, L, and slope, S, have been determined from the downstream node along the main 'side tributary' (see section B 1.1). Fixed values of both SAAR and SOIL have been applied throughout the catchment.
- As urban drainage can 'capture' runoff from across topographic divides, subcatchment boundaries to nodes have been determined by combining topographic and sewer-layout data. URBAN areas have been derived from digitised 1:10000 OS maps (not 1:63360 OS maps).
- Based on the derived 'catchment characteristics', the SCHEME model has estimated response parameters (Time to peak, channel lag, etc) using the equations of Flood Studies Supplementary Reports 5 and 16.

Land use upstream of the Cut at Binfield

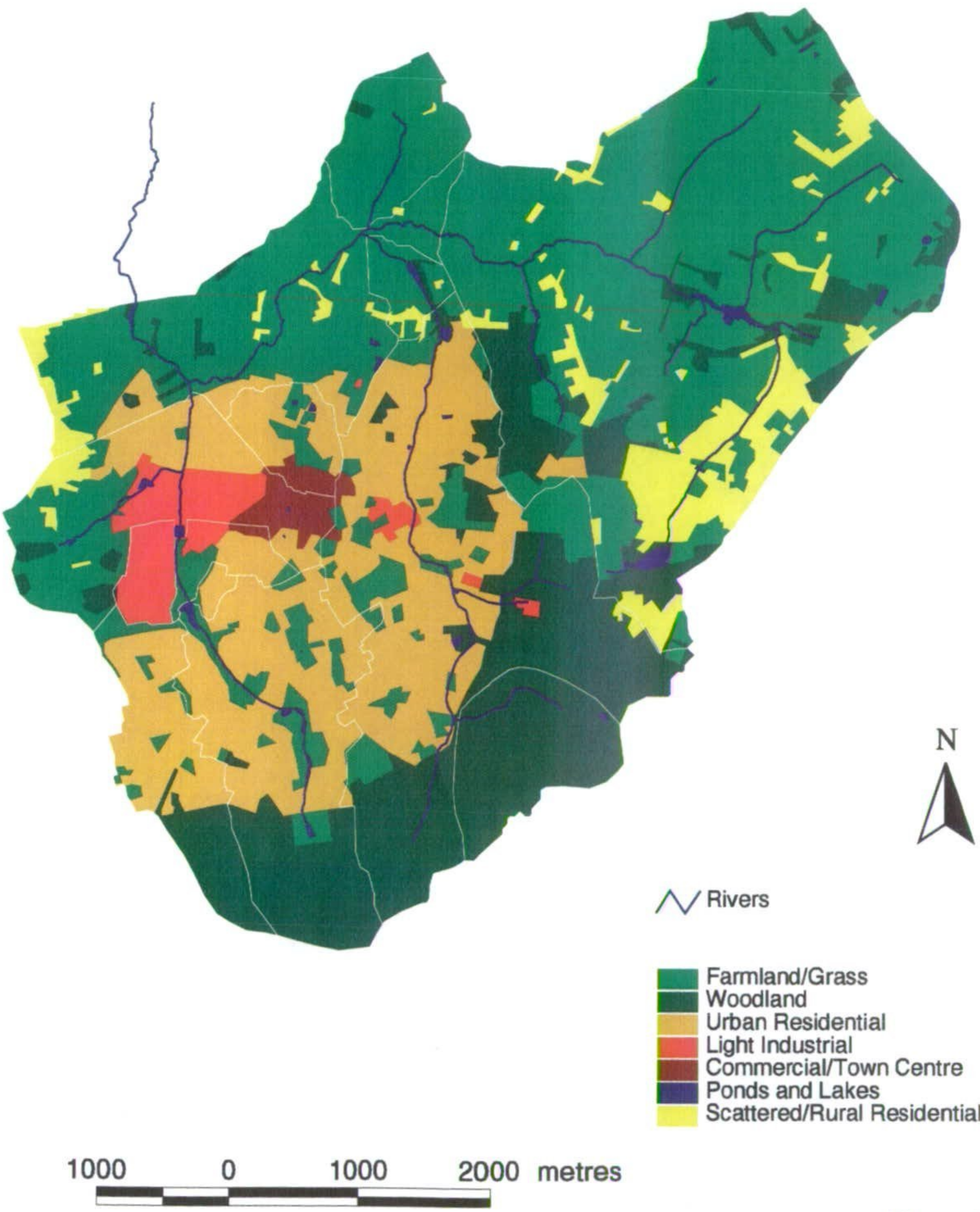


Figure B1

Modelled Catchments and Ponds

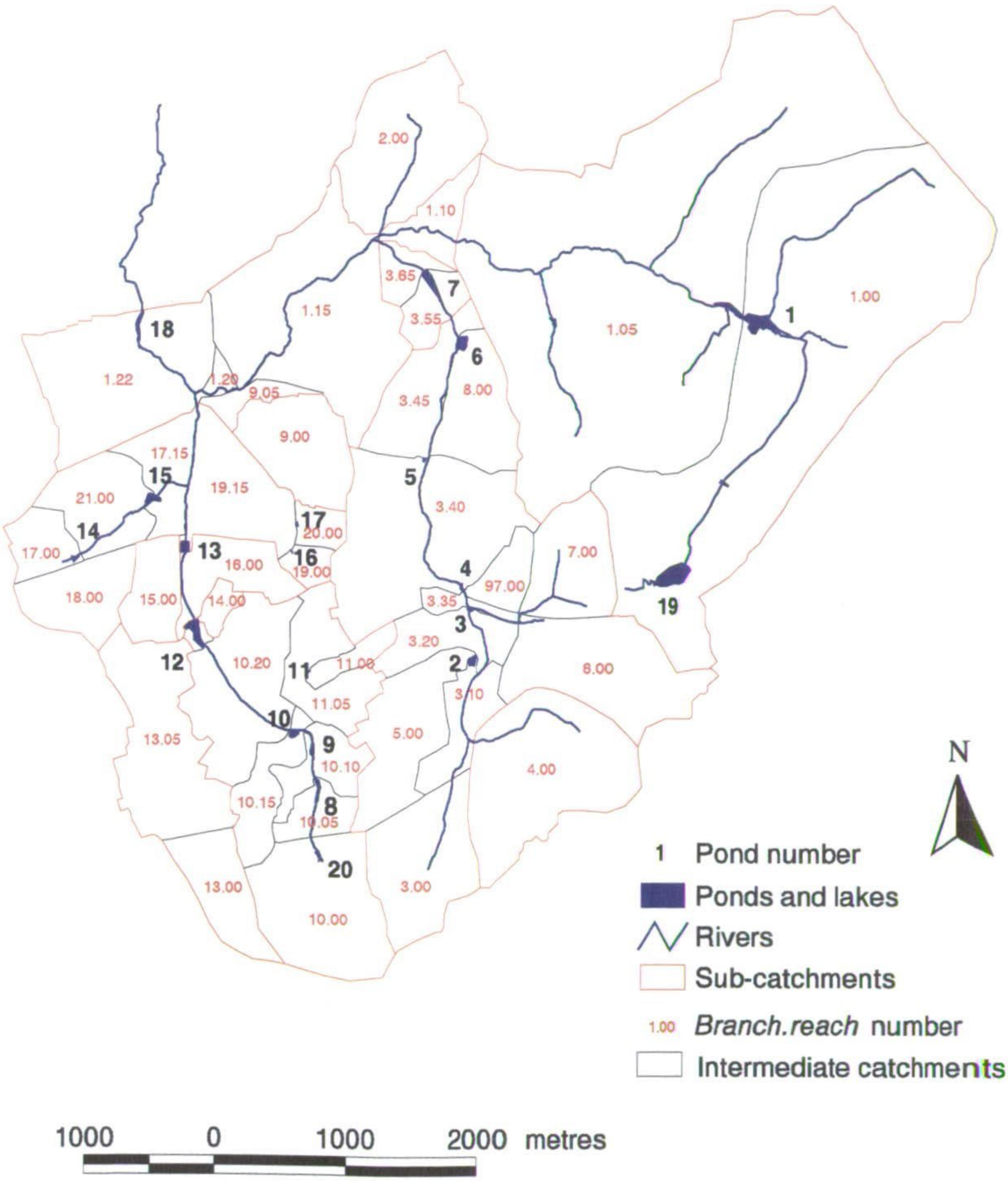


Figure B2

B 1.1 Sewer layout, drainage boundaries and catchment characteristics

Sewer layout data were needed:

- to gain a basic understanding of runoff from the urban area and to determine suitable monitor locations, and
- to define the exact drainage boundaries and flow paths within the urban area.

Sewer data were obtained from BFBC as computer datafiles for the 'STC25' program marketed by CDR. The information in these files is based on DoE/NWC(1980), and includes, for each manhole in the sewer network:

- the manhole reference, grid reference (to nearest metre) and system to which the manhole belongs (surface/foul/combined);
- the basic characteristics and dimensions of the manhole, its cover, shaft and chamber; and
- the shape, dimension, invert levels and destination manhole of all incoming and outgoing pipes.

The CDR program (and other similar competitor programs) allow:

- the details of any selected manhole to be retrieved or edited, and
- manhole and pipe layout to be displayed as long sections, plans, or maps plotted against a backdrop of (normally) the OS digital 1:1250 maps.

Although these maps do not include contour data, users can 'draw' drainage boundaries based on pipe layout and property boundaries and thus determine the surface and sewer data required by urban drainage models, such as HydroWorks. The 'vector' 1:1250 maps hold road and building outlines as separate 'layers', and their respective areas could potentially be derived automatically. However the polygons are not always closed, or indeed held as contiguous boundaries. Suitable processing software has now been developed, but at the start of the project the maps could only be used as backdrops.

The STC25 program together with OS 1:1250 maps covering Bracknell would have cost in excess of £20k, which was not justifiable for this research project. However, as the structure of the STC25 datafiles was known, a simpler program (STC) was developed to display and edit manhole details, to track up or downstream through the manholes, and to plot pipe layout maps of selected areas. This program, running on a laptop PC, was used on field visits, with printed OS maps, to seek out suitable manholes in which to monitor flow associated with an on-line and an off-line balancing pond, and areas of residential, commercial and industrial land-use. Locations were avoided that might cause unstable hydraulic conditions (due to junctions, bends or drops), or give access problems (deep sewers, busy roads, etc.). In Bracknell, the use of multiple bore pipes has often meant that more flow monitors would be needed than were available. It was largely for this reason that the original choice of which off-line pond to monitor (Bay Road in the Bull Brook catchment) was replaced with the Oldbury pond (in the Downmill Stream catchment), where the inputs could also double as industrial runoff monitors.

The STC25 datafiles as supplied by BFBC covered seven discrete areas (to meet size limits then present in the STC25 program), but they were combined into one area for the new STC program. A considerable number of errors were found, many relatively trivial (e.g. pipe diameter might be specified differently at upstream and downstream manholes), but others involving missing or bad links between manholes. It is not clear whether the errors had arisen because normal STC25 error checking had been disabled or because the data had been held as discrete areas. Having corrected the significant errors, the grid reference 'attributes' of each manhole and the ends of each pipe were transferred to the Institute's main GIS package, ARCINFO. Pipe layouts were then plotted against OS 1:10000 digital (raster) maps that had been obtained for the area. These maps are scanned from the original OS 'masters' at a pixel size equivalent to 0.6 metres on the ground. In raster form, and with all the text and symbols on one 'layer', they can only be used (at present) as a backdrop.

Although the printed OS 1:10000 maps show contours (in brown), this information has not been included in the digital product. To help define drainage boundaries, the ARCINFO plots were therefore overlain with other topographic data already held at IH, namely: the digitised 10m contours and river network from the OS 1:50000 maps; and selected drainage boundaries derived from the IH-Digital Terrain Model (DTM). A sample plot is given here (Fig. B3) but will also be referred to later.

The IH-DTM (Morris and Flavin, 1990) is defined on a 50m spatial grid, mainly by interpolation from the 10m contours. It is open to local re-interpretation where (i) either the original 10m contour interval or the 50 m spatial interval is inadequate, or (ii) catchment areas are captured by underground drainage systems. The sewer and road layouts on the 1:10000 maps have thus been used to adjust the DTM boundary in areas of conflict and to determine additional boundaries within the urban area. In this way, 42 subcatchments were defined for use in the SCHEME model. These have been shown on Fig. B2, and their areas (and other catchment data) are given later in Table B1. It may be noted that the topographic area of the Cut at Binfield is given as 50.2 km^2 in the National Water Archive and as 50.0 km^2 by the unadjusted IH-DTM. The figure of 51.9 km^2 shown in Table 3.2 of the main report includes forest and much of the housing area of Great Hollands (*branch.reach* 13.00 and 13.05) which topographically should not drain to the Cut but the Emm Brook.

In addition to catchment boundaries, the information shown on the OS 1:10000 maps has been used in a subjective assessment of land use. Within ARCINFO, digitised boundaries have been created for seven land use categories: farmland; woodland; rural-residential; urban-residential; commercial; industrial; and lake (see Fig. B1). The percentage of each land use in each of the 42 subcatchments was then defined. A summary for the gauged catchments is given in Table 3.1 of the main report.

The only land use included in the FSSR equations (used in SCHEME) is URBAN (the proportion of the catchment urbanised). This was originally defined as the 'grey area' on the OS 1:63360 maps (subsequently the 'pink area' on 1:50000 maps), and includes open pervious areas such as parks, sports fields and cemeteries. This definition can seldom be applied in small catchments or where development is planned. A more subjective assessment is made, generally taking the proportion of the area served by an urban drainage system, including domestic gardens, but excluding significant parks, etc. A more objective approach has been adopted for the forthcoming Flood Estimation Handbook (IH, in preparation), based on the extent of urban and suburban land use determined for 50 m x 50 m pixels using satellite images. This approach excludes park-land and large gardens. The FEH defines a new parameter, URBEXT as the proportion of urban pixels plus half the proportion of suburban pixels. URBEXT generally evaluates to about half the original FSR parameter URBAN.

In the current study, URBAN has been defined from the sum of the industrial, commercial, and urban-residential area, with half the rural-residential area. The exclusion of urban open space (see Fig. B1) may slightly underestimate the original FSR URBAN measure, but is considered a reasonable approach given the generally small subcatchments. The inclusion of half the rural residential development is also considered reasonable. In this catchment it has minimal impact on overall URBAN factors, except perhaps in subcatchment 1.00, where the Burleigh area has been classified as rural residential. House plots in this area are generally much larger than in Bracknell, and the area does not feel urbanised. Moreover, any URBAN impact will be well damped by the lake downstream at Ascot Place. URBAN factors for all the 42 subcatchments, together with the other catchment and channel characteristics, are given later in Table B1.

It may be noted that a single N-S flight line with an Airborne Thematic Mapper was also flown through the Bracknell town centre, but analysis to determine paved area was not pursued.

Having determined subcatchment boundaries and land use, representative stream length and slope were needed from which to derive subcatchment response times (using the FSSR16 equation). In the FSR (for a whole catchment), these are determined from the 'Main Stream'. However, apart from the uppermost subcatchment on a *branch*, the Main Stream does not truly represent surface runoff processes in a subcatchment. Length and slope are therefore defined for the largest 'side tributary', and will normally include only a part of the Main Stream (i.e. from the confluence with the side tributary to the downstream node).

The procedure is illustrated, within the urban area, using the Harmans Water subcatchment shown on Fig. B3. The old Main Stream (shown blue, running South to North) is now diverted through the Savernake pond, and thereafter runs in pipe (shown green). Thus, within the Harmans Water subcatchment (boundary shown black), the true main stream runs mostly in pipe for a distance of 560 m from the pond outlet to the sewer confluence about 100 m south of the railway line. The length and slope of this pipe form the 'channel length' and 'channel slope' used to route the upstream hydrograph. However, after 325 m this main stream meets a sewer tributary from the West, draining down through Harmans Water from a distance of 1244 m. This side tributary is taken as the subcatchment Main Stream. The subcatchment length (1244+560-325=1479 m) and slope (S1085) are then determined along this route.

The subcatchment areas, lengths, slopes and urban factors, plus the channel routing lengths and node heights used by SCHEME are given in Table B1 below. The table includes data for 42 subcatchments (shown on Fig. B2), together with 11 'channel only' reaches, all given in the *runoff calculation sequence* used by SCHEME. A schematic diagram showing the subcatchment and channel network has been given as Fig. 5.1 of the main report. In a few cases involving very small subcatchments (i.e. for 1.10, 3.55, 3.65, 1.20, 11.00) no 'Main Stream' could not be defined, and subcatchment length and slope have been estimated by considering map details and the values in neighbouring subcatchments. The channel width and roughness values are also basic estimates. The model is relatively insensitive to channel width, and as channel lag has been defined from FSR time to peak (see main report section 5.4.1), the Manning roughness values have not been used at all.

B 1.2 Storage ponds

In this study, eighteen storage ponds and lakes have been modelled (see Table B2). The three ornamental lakes modelled each have flood storage volumes comparable to the largest of the true balancing ponds (i.e. ponds specifically designed to control flooding), but have weir rather than throttle controls. Their storage is thus not necessarily well utilised in terms of reducing downstream flows. Note that on-line ponds route all upstream flows through the pond, while only high-flows are diverted into off-line ponds: low flows follow a bypass. Wet ponds contain water in low-flow conditions, while dry ponds drain completely. Table B2 also includes the *Branch.reach* code by which each pond is referred to in SCHEME, and which are shown in the schematic catchment diagram given as Fig. 5.1 of the main report.

Digital Coverages: sample area at Harmans Water

Figure B3

Table B1 Subcatchment data for SCHEME model of the Cut at Binfield

Branch reach	channel length km	node height m AOD	channel width m	channel Manning roughness	catchment area km^2	catchment stream length km	catchment stream slope m/km	urban factor
1.00		62.5	2		7.86	3.95	5.23	0.13
1.05	3.3	55.0	3	0.06	10.62	4.7	2.34	0.04
1.10	0.8	53.3	4	0.10	0.41	0.8	4.00	0
2.00		53.3			1.62	1.3	4.92	0
3.00		83.4			1.24	0.9	19.10	0
3.05	0.4	77.6	2	0.03				0.65
4.00		77.6			2.04	2.1	7.43	0
3.10	1.0	71.5	2	0.015	0.53	1.64	8.38	0.65
5.00		71.5			1.07	2.6	11.39	0.67
3.20	0.56	68.3	3	0.03	0.57	1.48	6.91	0.86
3.35	0.81	67.1	3	0.02	0.14	0.75	4.80	0.97
6.00		73.0			1.54	2.1	5.71	0.03
7.00		73.0			0.90	0.9	8.75	0.02
6.05	0.75	67.1	2	0.02				0.97
97.00		67.4			0.31	1.03	7.67	0.74
97.10	0.1	67.1	1	0.015				1
3.40	1.65	61.2	3	0.015	2.38	1.64	11.70	0.67
3.45	1.4	57.0	3	0.04	0.73	1.25	18.00	0.64
8.00		57.2			0.83	1.45	7.62	0.79
8.05	0.2	57.0	1	0.015				0
3.55	0.5	56.5	4	0.04	0.33	0.6	4.00	0.09
3.65	0.55	53.3	3	0.04	0.24	0.6	5.00	0
1.15	2.3	47.5	4	0.04	2.75	2.5	3.40	0.06
9.00		48.0			0.93	1.74	18.40	0.78
9.05	0.1	47.5	2	0.015	0.19	6.2	12.30	0.66
1.20	0.5	46.3	4	0.04	0.10	0.2	4.00	0
10.00		86.4			1.52	0.5	19.10	0
10.05	0.66	78.5	1.5	0.02	0.32	0.49	21.00	0.77
10.10	0.58	77.0	1	0.015	0.44	0.87	19.52	0.68
10.15	0.16	72.5	1	0.015	0.52	1.4	19.55	0.59
10.20	1.56	63.4	2	0.02	1.47	1.77	11.70	0.78
11.00		76.3			0.20	0.93	4.00	0.56
11.05	0.37	71.5	1	0.015	0.77	1.36	13.90	0.78
11.10	0.98	63.4	1	0.015				0.81
13.00		81.2			0.68	0.7	19.10	0
13.05	2.72	64.1	2	0.015	1.61	2.98	4.85	0.82
14.00		64.0			0.14	0.63	24.70	0.84
10.30	0.87	55.0	2	0.015				0.90
15.00		55.0			0.51	1.52	9.10	0.98
16.00		55.0			0.62	1.63	15.80	0.66
10.35	1.44	46.3	2	0.015				0.99
17.00		60.0			0.37	0.4	30.00	0.04
17.05	1.15	51.9	1	0.015				0.99
18.00		60.0			0.71	0.6	16.70	0
18.05	0.75	51.9	1	0.015				0.99
21.00		51.9			0.80	1.41	13.30	0.36
17.15	1.12	46.3	2	0.015	0.55	1.25	15.30	0.97
19.00		72.1			0.16	0.41	34.00	0.92
19.10	0.46	63.5	1	0.015				0.99
20.00		65.9			0.19	0.77	30.00	0.96
20.10	0.11	63.5	1	0.015				0.99
19.15	1.94	46.3	2	0.015	0.98	1.95	10.50	0.99
1.22	1.1	45.5	5	0.04	2.02	1.63	12.40	0.16

Table B2 Storage ponds and lakes.

No	Pond name	Branch.reach code	Pond type	Map Ref.	Pond Area (ha)	Flood storage $m^3 * 1000$
1	Ascot Place	1.03	ornamental	915712	5.50	27.5
2	Savernake Pond	3.17	on-line, wet	887678	1.02	18.5
3	The Warren	3.25	off-line, dry	886683	0.19	2.8
4	Martins Heron	97.05	on-line, wet	885686	0.77	9.3
5	Bay Road	3.42	off-line, dry	882698	0.25	2.6
6	Jiggs Lane	8.03	on-line, wet	884709	1.00	15.5
7	Warfield House	3.60	ornamental	882706	2.10	21.0
8	South Hill Park 1	10.08	on-line, wet	871667	0.69	2.4
9	South Hill Park 2	10.12	on-line, wet	870671	0.85	5.4
10	South Hill Park 3	10.18	off-line, dry	868671	0.57	2.7
11	Sports Centre	11.02	on-line, wet	870677	0.06	0.6
12	Mill Pond	10.25	on-line, wet	859682	2.63	27.0
13	Oldbury	15.02	off-line, dry	859690	1.05	8.6
14	Amen Corner	17.02	on-line, wet	848688	0.51	16.0
15	Waterside Park	21.05	on-line, wet	855695	1.34	25.9
16	St John's Ambulance	19.05	on-line, dry	868689	0.06	0.9
17	Multi-storey Car Park	20.05	on-line, dry	869692	0.08	3.0
18	Binfield Lake	1.25	ornamental	853712	2.10	21.0

The following sections present the information used to model these ponds and lakes. Most of the information was derived from:

SPS a Howard Humphreys and Partners (1988) 'Storage Pond Survey' (SPS) in the Cut catchment for Thames Water,

HV a number of Height:Volume survey reports obtained from EA Thames Region

These sources are indicated by their initials in the text below. Additional information was also obtained from plans and personal communication with BFBC, from reference to the STC25 database (see section B 1.1), and also from onsite measurements.

The Height:Volume surveys in **HV**, give storage volume at fixed intervals (normally 0.2 m), but the heights sometimes relate to an 'arbitrary datum'. Reasonable assumptions have been made as to what the datum might be. The bottom and top levels do not usually fit the 0.2m interval, thus implying (and sometimes stating) that the bottom levels refer to the Normal Water Level (NWL) of a wet pond, or control invert level of a dry pond, and the top levels may often refer to the emergency overflow or Top Water Level (TWL). The HV reports give tables of Volume against Height; but for SCHEME the data must be converted to Area against Height (where a linear change in Area corresponds to a quadratic change in Volume).

The data for the three ornamental lakes have been adopted as first estimates. A coefficient of discharge of 1.56 has been used to model all weirs. Notes of more specific assumptions or resolutions made in preparing the reservoir data are held at IH.

Although the ponds have been modelled as accurately as possible, it has not been possible to check the behaviour of any pond in detail. Considerable doubt persists over for example, the modelling of long side weirs. Improved modelling would be possible if more information became available, or if the effect of specific ponds needed to be assessed. Meanwhile, the modelling of the combined effect of all the ponds has been assessed through the accuracy of modelling river flows in the downstream network. While it is disappointing not to have more certain information on some of the ponds, it is unlikely that specific uncertainties will have had a great impact on the study conclusions. Within the time constraints of this study it has not been possible to confirm the effect of uncertainty through sensitivity analyses.

B 1.2.1 Ascot Place

This is an ornamental lake in the private grounds of Ascot Place. Photographs of the artificial rock control have been obtained from BFBC and have been used to help assess appropriate control dimensions. The lake area has been estimated as 5.5 ha from the 1:10000 digital maps. Thus a 0.5 m flood rise would give 27500 m^3 of flood storage.

The rock control has been modelled as a 4m broad crested weir ($Q=1.56*4*h^{1.5}$).
The storage area has been modelled as a constant 5.5 ha (i.e. no area growth with level).

B 1.2.2 Savernake Pond

This is an on-line wet pond in a landscaped park in a residential area near the top of the Bull Brook (see Fig. B3 in section B 1.1). It has a storage capacity (given by SPS) of approximately 18500 m^3. The outlet control is a rectangular orifice, with a second orifice and penstock, which has been assumed to be closed. When first constructed the pond was apparently on-line with respect to the main channel, but with an off-line input from a downstream tributary (pipe), via a side weir. However, the tributary now discharges directly into the pond. Pond volume:depth information is given by HV, but to an 'arbitrary datum'. There are considerable discrepancies between the level data shown on plans, as given in STC25, and as reported in SPS.

The outflow control has been modelled as a rectangular culvert (entry control), 900 mm wide by 200 mm high, at an invert level of 71.00 m AOD.

The emergency weir is 16.5 m long at a level of 73.25 m AOD.

The Height:Area data are:

H m	71.00	71.02	71.22	71.42	71.62	71.82	72.02	72.22	72.42	72.62	72.82	73.02	73.22
A ha	0.663	0.663	0.675	0.704	0.737	0.795	0.831	0.863	0.890	0.919	0.948	0.977	1.018

B 1.2.3 The Warren

This is an off-line dry pond just downstream of Savernake Park. It has a storage capacity (SPS) of approximately 1700 m^3, but creates another 1000 m^3 of storage by backing up in the Bull Brook. Downstream control is by three hydrobrakes in the Bull Brook, with flow diverted into storage via a side weir concealed in a bankside chamber. Drainage is by a flap valve below the side weir, and an emergency weir is provided above the side weir chamber. Pond volume:depth information is given by HV, but to an 'arbitrary datum'. The hydrobrakes are given as '542 mm type C' but no head discharge relationship was available. They have been modelled as three equivalent orifices of 380 mm.

Thus downstream control has been modelled as three 380 mm orifices at 69.35 m AOD.

The 2.5 m side weir has been modelled at a level of 70.35 m AOD.

The 250 mm flap valve is modelled at a level of 69.35 m AOD.

The 3.0 m emergency weir is modelled at a level of 70.65 m AOD.

The Height:Area data used are given as

H m	69.34	69 40	69.60	69.80	70.00	70.02	70.40	70.60	70.80	71.00
A ha	0.0	0.053	0.118	0.120	0.138	0.147	0.159	0.168	0.182	0.188

B 1.2.4 Martins Heron

This is an online 'formal' pond, with submerged inlets and outlets, beside a recreation ground (just visible on Fig. B3, section B 1.1). It lies on a surface water sewer, tributary to the Bull Brook. The flood storage is given in **SPS** as 9312 m^3 at a TWL of 70.10 m AOD, with the corresponding area of 0.7725 ha. For a NWL of 68.4 m AOD this implies a pond area of 0.325 ha. Outflow is by twin 450 mm pipes at an invert level of 67.40 m AOD, drowned by downstream weir at 68.40 m AOD. At low flows the weir is the control, but at higher flows the 450 mm pipes could act as throttles. STC25 also shows an alternative (emergency) outlet.

The downstream control has been modelled as a 2.5m weir at 68.4 m AOD, yielding to twin 450 mm orifices (with head set equal to head over weir).

An emergency outlet has been modelled as a 6.4 m weir at 70.10 m AOD.

The pond area is taken as 0.325 ha at 68.4 m AOD rising linearly to 0.7725 ha at 70.10 m AOD.

B 1.2.5 Bay Road

This is on off-line dry pond, providing flood storage of 2600 m^3 adjacent to the lower Bull Brook. Downstream control is by a combined underflow sluice and overflow weir structure across a 3.02 m channel, diverting excess flow into the pond via a long sideweir. The pond is drained by two 225 mm pipes with flap valves. Structure and layout details are given in **SPS**, and Volume:Height data are given in **HV**, related to a temporary bench mark.

Thus the downstream control has been modelled as a 3.02 m wide by 0.555 m high rectangular culvert (entry control) at 61.25 m AOD.

The 45.7 m side weir has been modelled at a level of 62.56 m AOD.

The 2 No. 250 mm flap valves are at a level of 61.25 m AOD.

The 3.02 m emergency weir is modelled at a level of 62.94 m AOD.

The Height:Area data used are

H m	61.25	61.30	61.50	61.70	61.90	62.10	62.30	62.50	62.70	62.90	62.95
A ha	0.0	0.006	0.017	0.019	0.1485	0.2115	0.2235	0.2315	0.2365	0.2425	0.2460

B 1.2.6 Jiggs Lane

This is a newly built on-line wet pond of flood storage 15550 m^3, adjacent to the Bull Brook and just upstream of Warfield House. Outlet control is by a hydrobrake installed in a bankside chamber, with emergency overflow direct to the Bull Brook via a 'grasscrete' depression in the bank. Design area and control details were supplied by BFBC.

The downstream control is given as a discharge table, where the transition phase of the hydrobrake is replaced with virtually constant discharge, and the emergency weir (data as supplied) is also held as a constant flow.

H m	57.2	57.3	57.4	57.5	57.6	57.7	58.2	58.4	58.6	58.8	59.8
Q m^3/s	0.0	0.017	0.040	0.064	0.096	0.139	0.140	0.150	1.96	6.8	6.8

The Height:Area data were given as

H m	57.2	57.4	57.6	57.8	58.0	58.2	58.4	58.6	58.8	59.0	60.0
A ha	0.0	0.6168	0.6656	0.7119	0.7504	0.7896	0.8311	0.8712	0.9431	1.0	1.0

B 1.2.7 Warfield House

This is an ornamental lake in the private grounds of Warfield House. The outflow weir dimensions have been supplied from EA Thames Region's survey records. The lake area has been estimated as 2.1 ha from the 1:10000 digital maps, giving 21000 m^3 of flood storage for a 1 m flood lift.

The weir has been modelled as a 4m wide weir ($Q=1.56*4*h^{1.5}$).

The storage area has been modelled as a constant 2.1 ha (no growth with level).

B 1.2.8 South Hill Park 1

This is a wet pond with a flood storage volume of 2400 m^3 in a park at the top of the Downmill Stream. SPS gives the outflow control as a 15 m wide weir at NWL, dropping into a transverse collector channel, 0.85 m wide then rising beyond to an emergency overflow weir. Flow from the collector channel drains through a throttle pipe, forming the normal flood control. A second outlet is assumed to be a scour/drawdown valve. HV survey details were not available, but SPS supplies the pond area at emergency weir level as 0.695 ha.

Thus the outflow control is a 15 m long weir at a level of 80.92 m AOD, giving way at higher levels to a 450 mm throttle pipe of invert level 79.66 m AOD.

The emergency weir is 18 m long at 81.26 m AOD.

The pond area is taken as a constant 0.695 ha irrespective of depth.

B 1.2.9 South Hill Park 2

This is a wet pond with a flood storage volume of 5350 m^3 in a recreation area just downstream from South Hill Park. SPS gives the outflow control as similar to South Hill Park 1, with an 18 m wide weir at NWL, dropping into a transverse collector channel, then rising beyond to an emergency overflow weir. Flow from the channel drains through a throttle pipe, forming the normal flood control. Again, HV survey details were not available, but SPS supplies the pond area at emergency weir level as 0.85 ha.

Thus the outflow control is an 18 m long weir at a level of 77.04 m AOD, giving way at higher levels to a 525 mm throttle pipe of inlet level 75.22 m AOD.

The emergency weir is 18 m long at 77.63 m AOD.

The pond area is taken as a constant 0.85 ha irrespective of depth.

B 1.2.10 South Hill Park 3

This is an off-line 'bog' area of flood storage 2670 m^3 almost directly downstream of South Hill Park pond 2. Downstream control is by a throttle pipe with flow into the pond controlled by a sideweir. The pond drains through flap valves and there is a (small) emergency pipe above the throttle pipe. Control details come from SPS/STC25, but Area:Depth data have been derived from plans supplied by BFBC.

Thus the downstream control has been modelled as a 525 mm throttle pipe (entry control) at 72.32 m AOD.

The 12.2 m side weir is at a level of 73.35 m AOD

The 2 No. 225 mm flap valves are at a level of 72.50 m AOD.

The 225 mm emergency weir is at a level of 73.59 m AOD.

The Height:Area data used are

H m	72.50	72.83	73.00	73.10	73.20	73.30	73.400	73.500
A ha	0.001	0.001	0.090	0.203	0.3130	0.410	0.490	0.570

B 1.2.11 Sports Centre

This is a small, on-line, wet (damp) pond with a flood storage of 620 m^3. Its impact on overall catchment response will be insignificant. It attenuates runoff from a sports complex, and has only been modelled in this study because the data were readily available. SPS gives the outlet as a low weir at NWL, leading to a 450 mm drain. 150 mm and 225 mm outlets are also present. Pond Volume:Height data are given in HV, related to an arbitrary datum (but with handwritten annotations to m AOD).

Thus the downstream control has been modelled as a 3.2 m weir at 75.05 m AOD, with throttle pipes of 150 mm at 74.82 m AOD and 225 mm at 74.76 m AOD.

The Height:Area data used are

H m	74.71	74.91	75.11	75.31	75.51	75.71	75.91	76.01
A ha	0.0	0.0217	0.0293	0.0347	0.0383	0.476	0.533	0.0585

B 1.2.12 Mill Pond

This is an on-line wet pond with a flood storage volume of 27000 m^3. It was once just a mill pond on the Downmill Stream, but was enlarged to balance urban runoff from a mostly residential area. It is the largest of the true balancing ponds in Bracknell. Monitoring of the inlets and outlets to the pond is described in section B 3.6. SPS gives the downstream control as a weir to retain NWL, discharging to a throttle pipe. Height:Volume data to OD are given by HV.

Thus the downstream control has been modelled as a 15.2 m weir at 62.65 m AOD, giving way at higher depths to two throttle pipes of 685 mm at 60.50 m AOD.

The emergency overflow is a 22.85 m weir at 63.76 m AOD.

The Height:Area data used are

H m	62.65	62.70	62.90	63.10	63.30	63.50	63.70
A ha	2.312	2.332	2.342	2.390	2.496	2.507	2.631

B 1.2.13 Oldbury

This is an off-line dry pond with a storage volume of 8600 m^3. Draining a mostly industrial area, the storage area was originally used as a horse paddock, but was subsequently redeveloped as a car-park on stilts (over a wasteland). As shown by SPS, the downstream control is a throttle pipe with diverted flow passing to the storage area via a long side weir. The throttled continuation flow joins the outflow from the Mill Pond (see above) before entering a culvert under a motorway link road and railway. The diverted flow travels approximately 60 m over rough ground to the storage oufall, where drainage is via two 200 mm pipes running separately under the road and railway. An emergency overflow weir discharges with the throttle and Mill Pond flow through the main culvert. Monitoring of the inflows and throttle (bypass) flows is described in section B 3.8, which includes photographs of the control arrangements. Pond Volume:Height data are given in HV, related to an arbitrary datum.

The downstream control has been modelled as a 1050 mm throttle pipe (entry control) at a level of 55.60 m AOD, at the end of a 3 m wide channel.

The 65 m side weir is at a level of 56.65 m AOD.

The 2 No. 200 mm drains are at a level of 55.0 m AOD.

The 6.556 m emergency weir is at a level of 56.885 m AOD.

The Height: Area data used are

H m	55.000	55.20	55.40	55.60	55.80	56.00	56.20	56.40
A ha	0.014	0.234	0.437	0.749	0.858	0.974	1.091	1.091

B 1.2.14 Amen Corner

This is an on-line, wet pond on an industrial park, with submerged inlets and outlets. The flood storage volume is 16000 m^3. Outflow is by twin 525 mm pipes at an invert level of 59.80 m AOD, drowned by a 2.8 m weir downstream at 60.30 m AOD. At low flows the weir is the control, but at higher flows the 525 mm pipes act as throttles. SPS gives TWL as 62.50 m AOD, but no details of an emergency overflow (a 5 m weir has been assumed). SPS gives pond area as 0.51 ha.

The downstream control has been modelled as a 2.8m weir at 60.30 m AOD, yielding to twin 525 mm orifices (with head set equal to head over weir).

An emergency outlet has been modelled as a 5 m weir at 62.50 m AOD.

The pond area is taken as 0.51 ha at all depths.

B 1.2.15 Waterside Park

This is an on-line, wet pond on an industrial park, with submerged inlets and outlets similar to Amen Corner. The flood storage volume is 25860 m^3. Outflow is by twin 250 mm pipes drowned by a 2.5 m weir downstream. At low flows the weir is the control, but at higher flows the 250 mm pipes act as throttles. SPS mentions an emergency overflow (a 5 m weir has again been assumed). Pond Volume:Height data are given in **HV**, related to an arbitrary datum, but the **SPS** values have been used in this study.

The downstream control has been modelled as a 2.5 m weir at 53.00 m AOD, yielding to twin 250 mm orifices (with head set equal to head over weir).

An emergency outlet has been modelled as a 5 m weir at 55.30 m AOD.

The pond area is taken as 0.9308 ha at 53.0 m rising linearly to 1.335 ha at 55.3 m AOD.

B 1.2.16 St. John's Ambulance

This is a small on-line dry tank with a flood storage of 887 m^3. Its impact on overall catchment response will be insignificant, and it has only been modelled in this study because the data were readily available. **SPS** gives the outlet as a 300 mm orifice, with an emergency overflow weir. Pond Volume:Height data are given in **HV**, related to an arbitrary datum.

The downstream control has been modelled as a 300 mm throttle pipe (entry control) at an invert level of 72.1 m AOD, with a 3 m overflow weir at 73.9 m AOD.

The Height:Area data used are

H m	72.10	72.30	72.40	72.70	73.90
A ha	0.001	0.001	0.0365	0.0555	0.0590

B 1.2.17 Multi-storey Car Park

This is an underground tank of 3000 m^3 within the sewer system, close to the town centre. It is the the only underground storage modelled in this study. Details and plans were supplied by BFBC.

The downstream control is a 375 mm throttle pipe at 64.17 m AOD, with an 675 mm overflow pipe at 66.80 m AOD.

The Height:Area data derived are

H m	64.17	64.37	64.51	68.08
A ha	0.0001	0.002	0.0810	0 0843

B 1.2.18 Binfield Lake

This is an ornamental lake in the private grounds of Binfield House. The outflow weir is Pitts Weir, used by EA Thames Region as a strategic flow station. The lake area has been estimated as 2.1 ha from the 1:10000 digital maps, giving 21000 m^3 of flood storage for a 1 m flood lift.

The weir has been modelled using the EA rating equation for Pitts Weir (see B 3.1 below).

The storage area has been modelled as a constant 2.1 ha (no growth with level).

B.2 RAINFALL DATA

The raingauges used in the study are listed in Table B3 and their locations are shown on Fig. B4. At the start of the study, existing recording raingauges covered the west side of the catchment, with the Met.Office meteorological site at Beaufort Park/Easthampstead (see marker a), and the EA telemetered flood warning site at Bracknell Sewage Treatment Works (see marker b). Additional sites were therefore needed in the centre and East of the catchment. Sites were chosen at: 3M gardens (c) in the centre of Bracknell; the Royal County of Berkshire Polo Club (d); Ascot Sewage Works (e); and Berkshire Golf Course (f). EA Thames Region installed Didcot 0.2 mm tipping bucket gauges at these sites between January and May 1993. All the gauges were conventionally installed with their lips 30 cm above ground level. For reasons of cost, recording gauge data were not obtained from the Beaufort Park gauge, but daily data for both Beaufort Park and Broadmoor Hospital (g) have been obtained.

Table B3 Raingauge details

Fig Ref	Raingauge site	Grid Ref:	Recording Gauge MO/EA Ref	Daily Check Gauge Ref.	First record
a	Beaufort Park, Easthampstead	SU846664	272735	272734	1978
b	Bracknell STW	SU858718	274918	274917	Jun 1986
c	Bracknell, 3M gardens	SU867696	926003	-	May 1993
d	Winkfield, RCB Polo Club	SU917719	926004	-	May 1993
e	Ascot STW	SU892683	926001	274917	Jan 1993
f	Berkshire Golf Course	SU903659	926002	283677	Jan 1993
g	Broadmoor Hospital	SU852641		271567	

Rainfall data from the EA recording raingauges were obtained as tip times to the nearest minute (five minutes at Bracknell Sewage Works, part of the EA telemetered raingauge network). In most hydrological studies rainfall intensities would be determined by assuming

Hydrological monitoring network

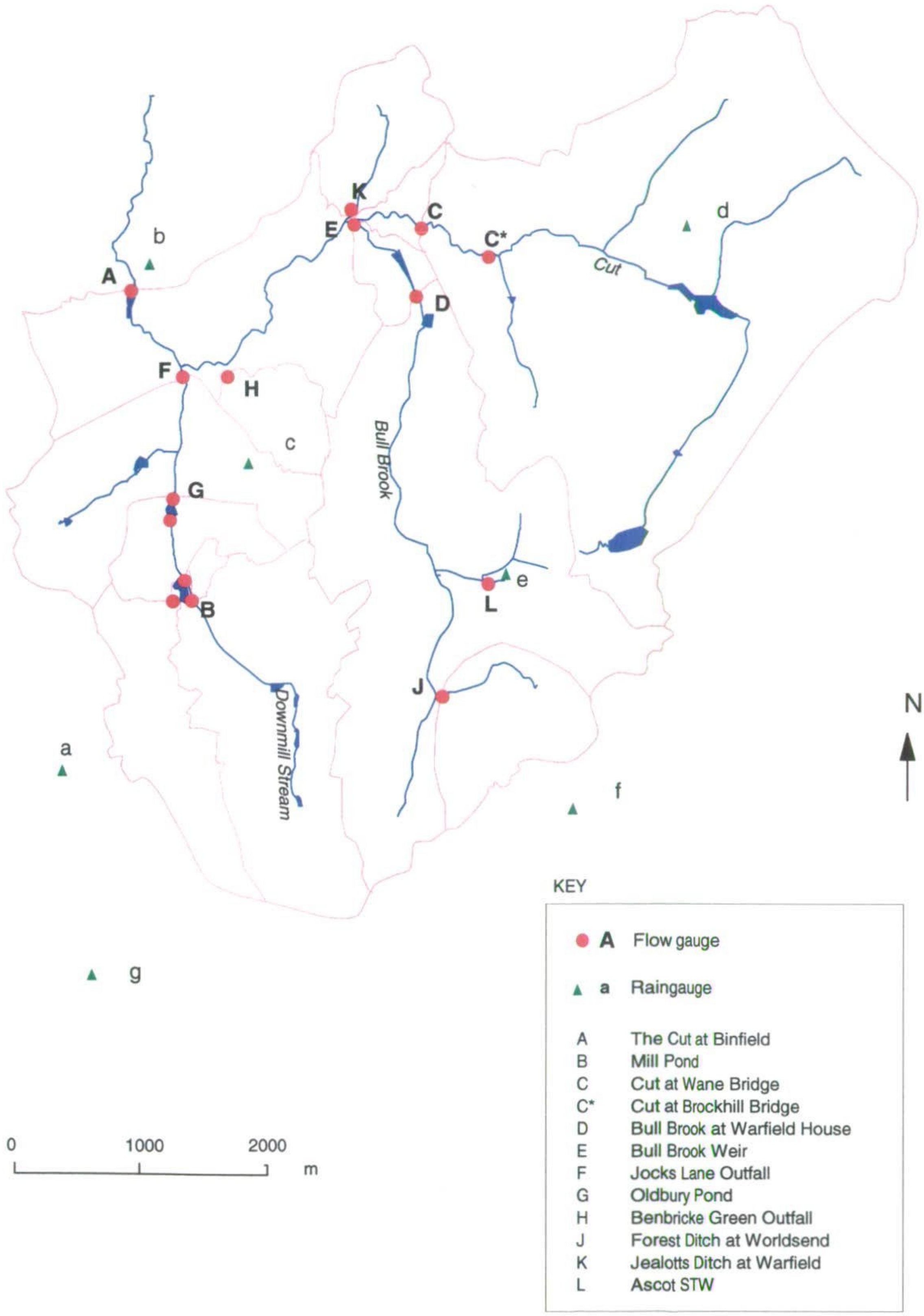


Figure B4

that the tips occur at the recorded time, and that all the tip volume fell during the previous interval (the tip actually occurs during the interval, and the bucket may be part full at the start of the interval). The on-off nature of these assumptions is usually smoothed by accumulating the tips over a model interval of 15 minutes to 1 hour. In this study, with rapidly responding catchments, and concern for storms of moderate total depth but high intensity over short periods, a 5-minute model interval was required (sewer surveys typically adopt an interval of 2 to 5 minutes). Short accumulation times tend to yield rainfall profiles that are very 'square wave' in form. This is presentationally unattractive (hyetograph plots look odd) and may also affect model performance. An improved processing algorithm has therefore been developed, following an approach adopted for the development of WASSP.

The algorithm assumes:

(1) Raingauge 'tips' are uniformly distributed over the recording interval (i.e. over 1-minute for the 'new' gauges, 5-minutes for Bracknell STW). Thus a single tip is assumed to occur midway through the interval, two tips would occur at the quarter and three quarter points, etc.

(2) Rainfall spans 'dry intervals' until the implied intensity falls below a critical limit (set to 0.5 mm/h). Thus for a 0.2 mm bucket and a 1-minute recording interval, tips less that 24 minutes apart are treated as continuous rainfall. For tips further apart the rain is deemed to have stopped, and is formed into bursts.

(3) The raingauge bucket is half full at the start and end of each burst. Thus 'true' start and end times of rainfall are extrapolated from first and last tip times by half the time between the first/last <u>two</u> tips (i.e. by the time to half fill the bucket).

Distribution of gauged point rainfall depths and profiles to the various subcatchments used in the modelling study was performed within SCHEME (see Appendix A).

B.3 CHANNEL FLOW/LEVEL DATA

Details of the location and type of flow gauges used in this study are summarised in Table B4 and Fig. B4. Table B5 describes briefly the different instruments used. The general approach to processing velocity data from flow monitors is described below, and subsequent sections present particular issues related to specific gauges. Where possible data have been collected at a 5-minute timestep, except for certain sites managed by EA Thames Region, where data needed to conform to their standard 15-minute timestep.

Table B4 Gauge locations, descriptions, instrument type and data periods

Fig ref	Gauge Name	OS Grid Ref	Description	Type of Instrument	Period of Data
A	Binfield	SU853713	13.7m wide short crested weir, 1.2m low flow section	Level Recorder	1957 - 1997 (digital 1986..)
B_1	Easthampstead inlet	SU857680	Rectangular flume, 1.785m throat, 2.73m approach	Technolog Logger	1993-1997
B_2	Great Hollands inlet	SU856680	Rectangular flume, 1.21m throat, 1.83m approach	Technolog Logger	1993-1997
B_3	Wildridings inlet	SU859682	Rectangular flume, 0.755m throat, 1.22m approach	Technolog Logger	1993-1997 (incomplete)
G_4	Outlet (at Oldbury Pond)	SU858689	1350 mm pipe	Detectronic then Water-Rat	1993-1996 1996-1997
C	Wane Bridge	SU884719	Bridge opening	Level recorder	1986-1997
C*	Brockhill	SU892716	Rectangular Bridge opening 3.05 m wide	Starflow	1995-1997
D	Warfield House	SU884712	Rectangular Bridge opening, 5.18 m wide	Technolog Logger and Starflow	1986-1997 and 1995-1997
E	Bull Brook Weir	SU883711	Broad crested weir, 2.16m wide	Level Recorder	1986-1997
F	Jocks Lane	SU859703	Triple 1800 mm pipes	ADS Meters	1993-1997
G_1	Industrial inlet	SU858689	1300 mm pipe	Detectronic	1993-1997
G_2	Waitrose inlet	SU858689	1300 mm pipe	Detectronic	1993-1997
G_3	Bypass outlet	SU858689	1050 mm pipe	Detectronic	1993-1997
H	Benbricke Green	SU863703	Twin 1050 mm pipes	Detectronic	1993-1997
J	Worldsend	SU886671	585 mm culvert	Water-Rat then Starflow	1995-1997 (incomplete)
K	Jealotts Ditch	SU877720	750 mm road culvert	Water-Rat	1995-1997 (incomplete)
L	Ascot STW	SU892683	Rectangular thin plate weir, 2.01 m wide	Technolog Logger	1996-1997

Table B5 Instrument details

Name of Instrument	Measurement of Depth/Level	Measurement of Velocity
Level Recorder	Float and Stilling Well	Not Measured
Technolog Logger	Pressure Transducer	Not Measured
Prolec Water Rat	Pressure Transducer	Ultrasonic Doppler
Unidata Starflow	Pressure Transducer	Ultrasonic Doppler
Detectronic (Montec)	Pressure Transducer	Ultrasonic Doppler
ADS	Downward Ultrasonic and Pressure Transducer	Ultrasonic Doppler

As is evident from the tables, this project has made wide use of ultrasonic-Doppler meters. Data recorded with these meters can contain periods of erroneous velocity and varying amounts of noise. The reasons include uneven velocity distribution, too few 'targets' in the flow to reflect the ultrasonic beam, and masking or 'ragging' of the sensor head. Developing procedures to identify and correct bad velocities, and to smooth out noise has been a major component of this study. The approach adopted has been to study the time series of 'conveyance', and apply a moving average (usually of 3-points) to reduce the noise yet conserve the overall total flow. On occasions, notably for summer storms at Brockhill Bridge, a 9-point moving average was applied. These storms yielded low velocities with a high degree of noise, and the slow response of the catchment meant that applying a longer moving average did not hide the storm response. To quantify the differences in the steadiness of the depth and velocity signals, the lag-1 auto-correlation statistic was calculated for each series. Other statistics, including residual variance and mean deviation from moving average gave simple indications of the variability of the data and the degree of smoothing that had been applied.

Occasionally, the velocity was registered as negative. Although the sensors are capable of recording reverse flow, the values often occurred during a storm period when previous and subsequent readings indicated a good flow of water. When this occurred, the negative value was replaced with a velocity calculated from the corresponding depth using the average conveyance.

During low flows, with a poor signal return, Starflows would occasionally record velocities of between 2 and 4 m/s, a flow-speed unrealistic for such depths. The instrument's firmware also allowed velocity recordings to 'stick' at these high values, giving data that was clearly erroneous. In these cases, data values were re-evaluated using depth and the average conveyance. The same procedure was employed when zero or unexpectedly low values were obtained, or if sudden changes in velocity indicated that a problem existed with at least one of the readings. At the same time, by examining the march of conveyance values, periods of low velocity due to backing up were preserved.

The procedure adopted to calculate flows at the Doppler meters was thus as follows:

(1) The hydraulic radius, RAD, was calculated as:

$$RAD = AREA/PERIM \qquad (B1)$$

where AREA is the area of flow and PERIM is the wetted perimeter, calculated from the recorded depth and channel shape and dimensions.

(2) Noting that the Manning equation for flow velocity, V is:

$$V = (1/n)\, RAD^{2/3}\, S^{1/2} \qquad (B2)$$

where S is the channel slope and n the Manning's roughness coefficient

The channel conveyance $CONVEY=(1/n)S^{1/2}$ was derived for each observed depth and velocity pair as

$$CONVEY = V/(RAD^{2/3}) \qquad (B3)$$

(3) The time series of conveyance values was studied to identify outliers and substitute averages from the filtered data set. The time series was then smoothed as appropriate, using a moving average, and new velocity values found as:

$$\text{VELCALC} = (\text{RAD}^{2/3})\text{CONVEY} \qquad \text{(B4)}$$

(4) Finally, the corresponding flow, for each depth and velocity data pair, was then determined as:

$$\text{QFLOW} = \text{VELCALC} * \text{AREA} \qquad \text{(B5)}$$

The remainder of this part of the appendix now turns to description of specific gauges and any issues arising. Photographs of many of the gauges are given in the Plates at the end of the text.

B 3.1 The Cut at Binfield (EA No. 2620)

Located at SU853713 just North of Pitt's bridge, this gauge is a short crested weir on the outfall of an ornamental lake. It has a drawdown sluice, the crest of which has been lowered below the main weir to provide a low flow control (see Plate B1). The sluice collects debris casting some doubt on low flow data, but the effect is probably quite small. Level data are collected at a 15-minute timestep by the EA as part of their strategic network, and Mean Daily Flows are included in the National Water Archive (station number 39052). The data obtained for this project are (a) monthly maxima and peaks over threshold (3.8 m^3/s) since 1957, and (b) 15-minute levels/flows since 1986.

The mean average flow over the weir between January 1987 and December 1996 was 0.40 m^3/s. Yearly instantaneous maxima for 1987 to 1996 ranged from 12.77 m^3/s down to 5.58 m^3/s. With regard to the monthly maxima collected since 1957, the maximum value was recorded in June 1981 at 18.1 m^3/s and the lowest monthly maximum was 0.11 m^3/s measured in October 1978.

The stage discharge relationship for the Cut at Binfield is

$Q = 2.524\, H^{1.5955}$	$H < 0.168$m
$Q = 137.05\, H^{3.832}$	$0.168 < H < 0.274$m
$Q = 36.382\, H^{2.807}$	$0.274 < H < 0.411$m
$Q = 19.416\, H^{2.100}$	$0.411 < H < 0.671$m
$Q = 18.110\, H^{1.926}$	$H > 0.671$m

where Q = discharge (m^3/s), and
H = water level above crest of weir or sluice (m)

Figure 3.7(d) in the main report shows the data collected during this study period, and Fig. B5(d) shows the data for June 1994, where the diurnal level variation due to Ascot sewage works is just discernable.

B 3.2 Bull Brook at Warfield House (EA No. 2606)

This gauge comprises a pressure transmitter with Technolog logger installed by a stilling tube on the upstream bankside of the bridge at SU884712. Figure B6 shows the gauge location (D) together with the locations of Bull Brook Weir (E) and Wane Bridge (C) - see below. The stilling tube was originally installed for a float gauge, but is silted up, so the transmitter was led out into the channel and simply weighted down with a piece of concrete. Plate B2 shows

June 1994: data from principal gauges

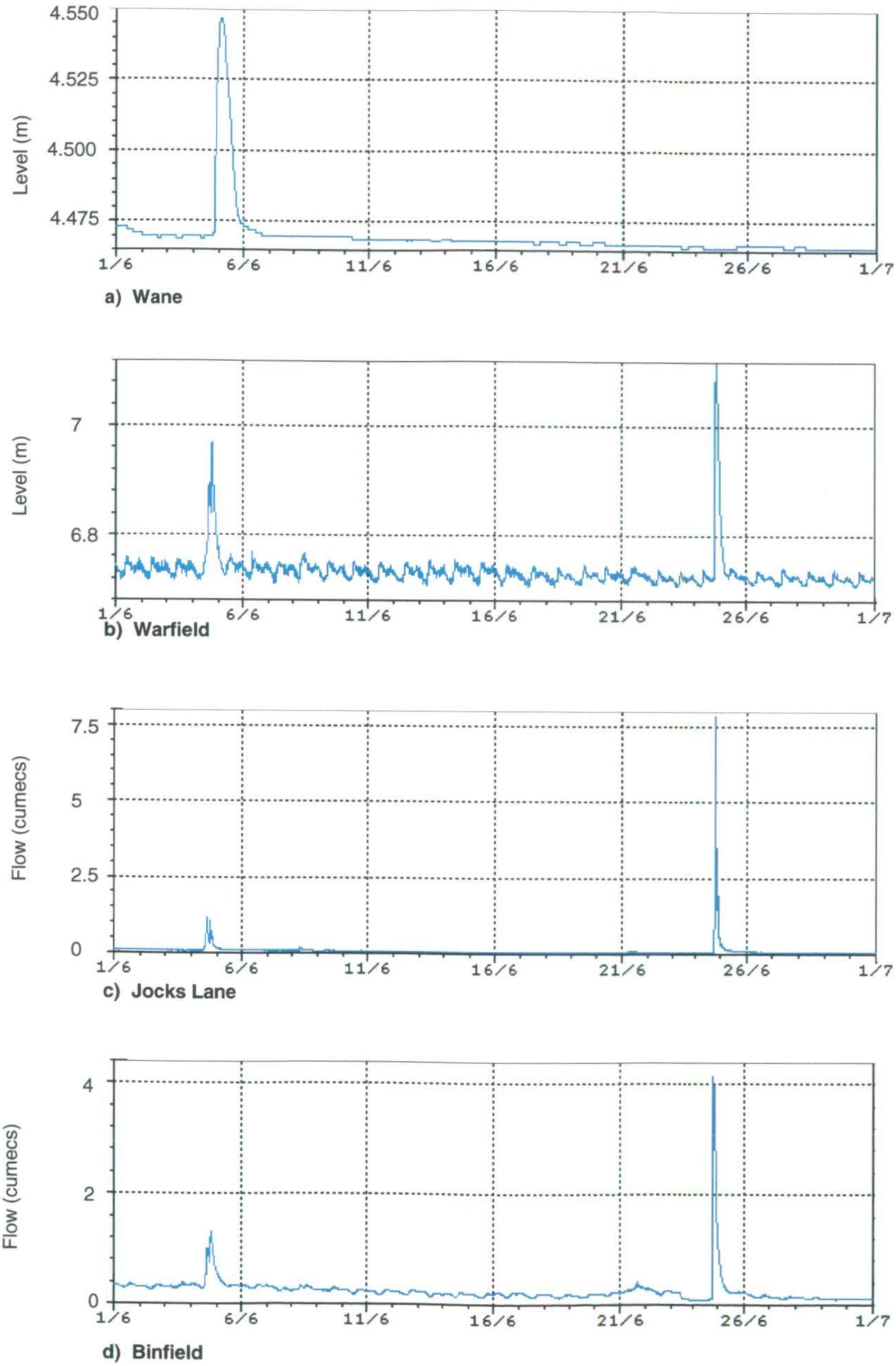


Figure B5

Gauge Locations: Warfield House (D), Bull Brook Weir (E), Wane Bridge (C), Brockhill (C*) and Jealotts Ditch (K)

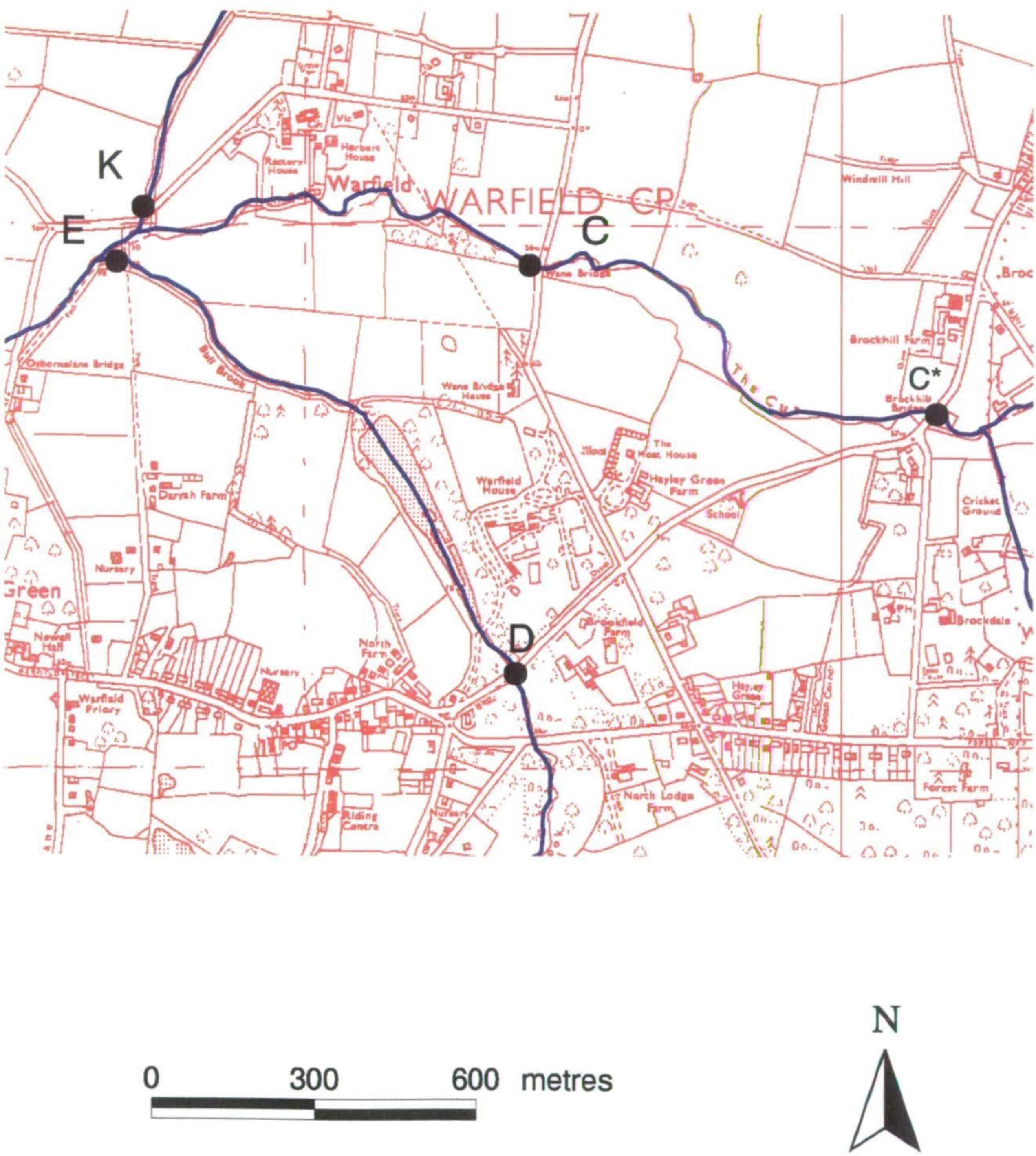


Figure B6

the channel looking downstream towards the gauge. Figure 3.7(b) in the main report shows the data collected during this study period, and Fig. B5(b) shows the data for June 1994; the diurnal level variation due to Ascot sewage works is clearly visible.

B 3.2.1 Establishing the Rating Equation

Level data, at 15 minute intervals, have been collected by the EA since 1988, but there are considerable gaps (e.g. Oct. 1991 to Mar. 1993) and major changes in flow conditions. Under the bridge, the channel section is essentially rectangular, comprising a concrete bed between brick sidewalls 5.18 m apart. However below 500 mm depth the profile is complicated by an earth bank deposit and a bolster. The effect on the channel profile is shown in Fig. B7a (though in reality the bank and bolster are on opposite sides of the channel). To establish the rating equation, a Starflow instrument was installed on the concrete channel bed, under the mid point of the bridge, in February 1995 (though satisfactory data were not obtained until the summer of 1995). Readings of depth and velocity were logged at 5-minute intervals and flows were calculated from the channel profile. Check gaugings have been performed at the site, but conversions from point velocity readings at the Starflow to section average have not been made.

B 3.2.2 Relationship between measured stage and depth of flow

As the first stage in developing a rating, it was necessary to check the consistency of the Starflow depth (timestep 5 minutes) against the EA stage data (timestep 15 minutes). Values for each were compared during the period when both were in operation. A graph of depth against stage should have yielded a straight line with limited scatter caused by the loggers recording at slightly different times. Such discrepancies would be most apparent during the rising limb of a storm when the depth was changing most quickly; the two instruments would be unlikely to record at exactly the same time.

From the plot (Fig. B7b), it is clear that during one storm event (9 Aug 1996), at least one of the instruments was faulty. This event was not included when calculating the relationship between Starflow depth and stage. Also, the data for July 1995 show considerable scatter probably due to the two instruments taking measurements at slightly different times, which, due to the exceptionally rapidly rising and falling limbs resulting from the high summer storm intensity, caused a greater than usual discrepancy in the results.

Figure B7c shows the remaining data points after removing the data for the storms of 9 August 1996, 5 July 1996, 26 July 1995 and 27 July 1995. There is very little scatter in the data enabling greater confidence when calculating the water depth from the stage for the period before the installation of the Starflow. The resulting equation for converting stage, y (m) to depth, d (mm) is

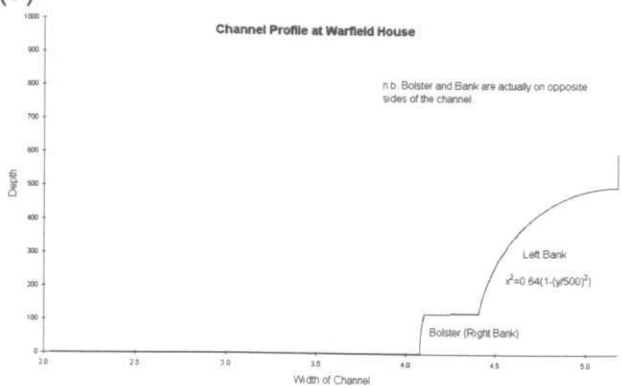
$$d = 1000y - 6641$$

B 3.2.3 Rating Equation

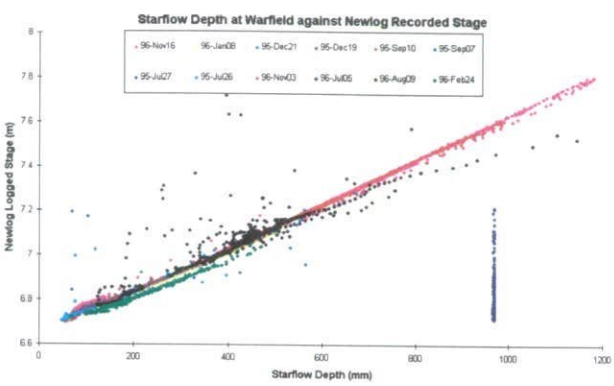
Having confirmed the consistency between stage and depth readings, it was necessary to establish a rating equation using the Starflow data. Depth values were used, along with the channel dimensions, to calculate the cross-sectional area of flow for each logged depth. Multiplying this value by the velocity yields flow values and an equation describing the relationship between depth and flow was derived. Combined with use of the equation relating stage and depth, this can be used to create flow data from stage values. This method relies on

Graphs for Warfield House and Bull Brook Weir

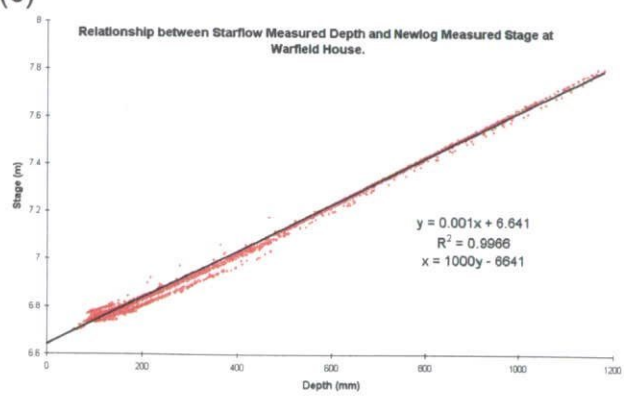
(a)



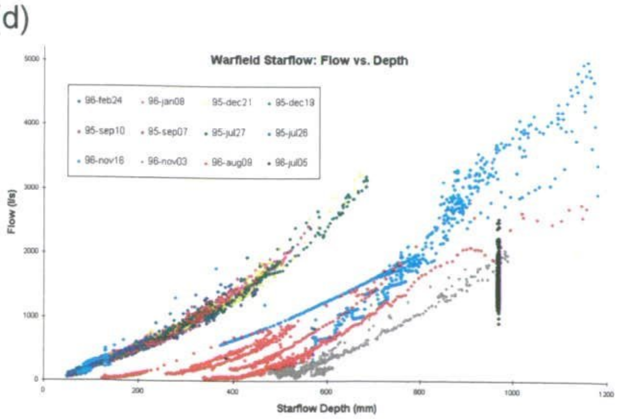
(b)



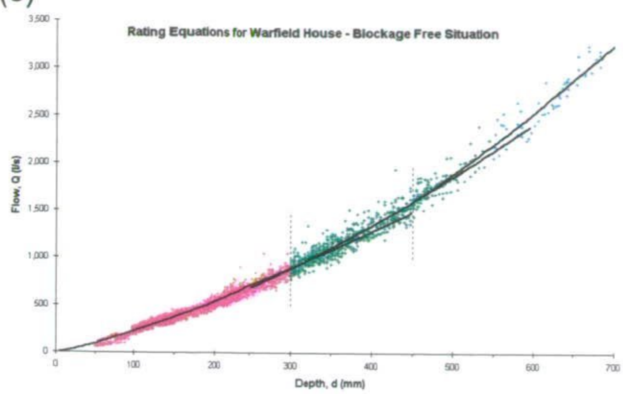
(c)



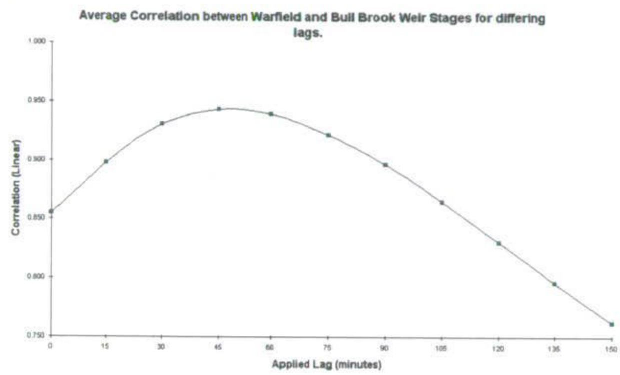
(d)



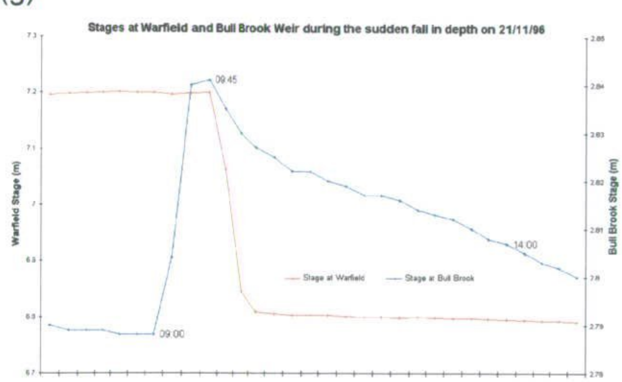
(e)



(f)



(g)



(h)

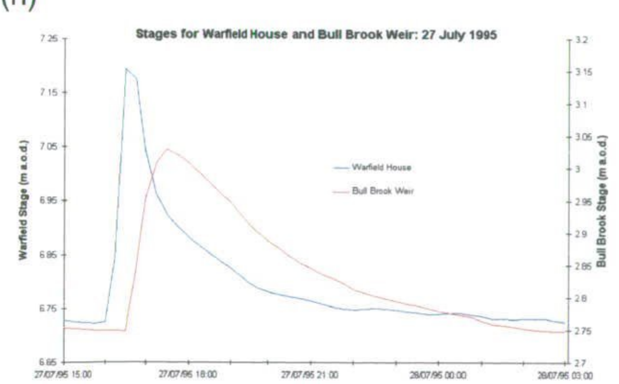


Figure B7

the flow characteristics of the site remaining constant over time, which is not always the case, as discussed below.

For each of the 12 storms with Starflow data available between July 1995 and November 1996, flow was plotted against depth as shown in Fig. B7d. It was hoped that all data points, from all storms, would lie approximately on a single curve. This would confirm that the flow characteristics at Warfield House were constant and a certain depth would correspond to the same flow at any time in the past. This clearly is not the case with four storms having wildly different curves to the others. It appears that up to and including the storm of 24 February 1996, the channel had a steady rating. However, after this time lower velocities, and thus flows, are recorded for certain depths than were before. This might have been due to instrument failure, but since the relationship between stage and depth remains constant for the whole period (except for the storm of 5 July 1996 when the pressure transducer was reading incorrectly) it seems this was not the case. Rather, it appears that a blockage downstream caused the flow to back up, resulting in increased depths and decreased velocities, and it also seems that the blockage was cleared during the large rainfall event beginning on 16 November 1996, since the depth suddenly dropped towards the end of the storm and the last values for this event lie on the curve of the first eight storms. This suggests that the flow conditions demonstrated by the earlier eight storms represent a 'ground' or 'steady' state with deviation from this rating being temporary. Thus it is possible to apply the rating obtained from studying data from only the first eight storms to previous data, provided care is taken to exclude previous periods when blockages occurred.

In order to detect the periods when applying a rating would be valid the whole level record (July 1993 to August 1996) from the level sensor was plotted. This clearly showed that there was a period from 12 December 1993 to 16 February 1994 when the channel was blocked and depths were significantly augmented. It is evident that the related flows were not significantly higher than during other periods, since the depths never dropped to a low level during this period and were clearly maintained not by upstream inflow but by a downstream control. There was a sudden drop in water level in February 1994 of approximately 500 mm which seems to be attributable to debris being cleared away, either by the force of water or deliberately.

The maximum stage occurring during the eight storms used in deriving the rating was 7.294 m (depth = 633 mm). Previous storms, to which we wished to apply the rating, resulted in higher maximum stages of 7.905 (d=1264 m) and 7.620 (d=979 mm). Before applying the rating to these storms we needed to be sure that the increased stage was due to increased flow and not due to a blockage or other downstream control. The high stages of 7.905 m and 7.620 m occurred in the Autumn of 1994. Although the peak stages for the two events were exceptionally high, the water level quickly returned to a level close to that of dry weather flow, indicating a free flow of water and no backing up. It would thus appear that the rating curve derived earlier can realistically be applied to any storms not in the period 12 December 1993 to 16 February 1994.

The eight storms lying on the same curve were plotted and the rating curve was derived as a power law. In order to achieve a good fit, three different equations were applied, each one relevant to a different range of depths as follows and in Fig. B7e. (Values of the coefficient of determination, R^2, are also given to indicate the strength of the relationship)

$$Q = 0.6855d^{1.2576} \qquad d < 300mm \qquad (R^2 = 0.9532)$$
$$Q = 0.2279d^{1.4493} \qquad 300mm < d < 450mm \qquad (R^2 = 0.9219)$$
$$Q = 0.0833d^{1.6138} \qquad d \geq 450mm \qquad (R^2 = 0.8292)$$

where Q = discharge (m^3/s) and d = water depth (mm)

B 3.3 Bull Brook Weir (EA no. 2608)

This gauge at SU877719 is just upstream of the confluence of the Bull Brook with the Cut (see Fig. B6 marker E) and not far downstream of the gauge at Warfield House (discussed in section B 3.2). Between the two gauges lies the ornamental lake at Warfield House, the effect of which is to dampen the data at the downstream gauge. The site comprises a broad crested weir, stilling well and shaft encoder, with data logged at 15 minute intervals. There are gaps and transmitter drift present in the record, notably for the storm of 12 October 1993 when the level backed up from the confluence with the Bull Brook and became so great that the float came detached from the shaft encoder. Despite the presence of a weir, a rating equation has not been derived for the site because:

- the Warfield House gauge upstream was closer to the urban area and was not subject to the effect of the lake
- the approach channel to the weir is poorly defined, overgrown, and silted, creating more of a drop structure than a weir, and the approach flow appeared to be supercritical (it 'babbled')
- the weir drowns in high flows.

However the data were examined to try and explain rating changes at Warfield House.

The level data at Warfield House and at Bull Brook Weir were compared for 29 storms and correlation coefficients were calculated for a range of lag times applied in 15 minute steps. This showed that the intervening ornamental pond and approximately 0.5 km of natural channel delayed the flood peak by an average of about 45 minutes (see Fig. B7f). By examining each storm in turn it was interesting to note that the minimum lag time (i.e. that which gave maximum correlation) was 30 minutes and the maximum was 1 hour. All but two of the storms revealed lags of either 45 minutes or 1 hour (disregarding those storms when the channel at Warfield was backed up). Considering the variability in the storm magnitude and hydrograph shape this consistency in lag time between Warfield House and the weir is noteworthy.

It is clear that the cause of the backing up which affects the data record at Warfield House (see section B 3.2) must lie between the gauge at Warfield House and that at Bull Brook Weir Site. Figure B7g shows the level data at each gauge for 21 November 1996 when the 'blockage' cleared at approximately 09:00. The stage at Bull Brook is the first to be affected, the level rising suddenly as the water held upstream was released. The level then falls gradually over more than 5 hours as the stored water drains away. At the Warfield House gauge, upstream of the blockage, the response comes later, with a sudden drop in level as the effect of the blockage is removed. Figure B7h shows the level hydrographs at Warfield House and Bull Brook Weir for a more typical event of 27 July 1995. The lag and attenuation between the two gauges are both clearly visible.

B 3.4 The Cut at Wane Bridge (EA no. 2612) and Brockhill

At SU884719, this gauge comprises a pressure transmitter in a stilling tube on the downstream bankside of Wane Bridge (see Fig. B6, marker C). Level data are available at 15-minute intervals reasonably consistently from 1988, but siltation has occurred around the stilling tube and there is some apparent transmitter drift. Plate B2 shows the river downstream of the gauge during the flood event of 12 October 1993. The river is well out of bank on the left side, with the normal bankside identified by the section of fence in the nearground. The river bed under the bridge is concrete, but the river bends upstream, and there is a mid-span pier for the upstream half of the bridge. The site is adequate for monitoring level but not really suitable for developing a channel rating. A rating was therefore sought using another site nearby.

As at Warfield a channel rating was needed to convert the available level data to discharge. Level data were available for all storm events examined from 1993 to 1997, except for the storm event of 7-10th March 1995. The Brockhill Bridge site was selected to develop the rating, approximately 800 m upstream and with no major intervening inflows. A Starflow instrument was installed to provide depth and velocity readings at 5-minute intervals.

B 3.4.1 Brockhill: Channel Cross-Section and River Characteristics

The Starflow was located under a bridge where the river cross-section is a 3050 mm wide rectangular channel. It was expected that this would give a reasonably uniform velocity distribution across the flow, though a bend upstream of the bridge and some shoaling downstream would cause some variation. With the Starflow located centrally this may have meant that the velocities it measured were lower than the channel average, especially during periods of higher flow when higher velocity differences would have existed across the channel width. Some current meterings were made across the profile, but no adjustments to the Starflow velocities were made.

B 3.4.2 Lag Time

It was necessary to establish the lag time associated with the distance between the Brockhill and Wane Bridges, so that flow depths at Brockhill could be compared with stages at Wane. The correlation coefficient between Brockhill depth and Wane level was calculated for each storm, for a range of different lag times, ranging from zero to 5 hours in 5-minute steps. 15-minute levels at Wane Bridge were linearly interpolated in order to create 5-minute level approximations. The correlation was seen to peak at a certain lag time for each storm period and this peak defined the lag time for the storm.

The correlation plots (Fig. B8a) indicate that in winter the lag time between Brockhill and Wane Bridge depths was approximately 50 minutes, but in the summer was longer, increasing to over four hours in September. This could be due to summer vegetation growth reducing the channel conveyance. However, as the lag for the August storm was only 85 minutes it seems that lag time also depends on event magnitude. Table B6 below shows lag times for each storm as well as maximum depths/stages and flows. Figure B8b is a plot of the maximum stage at Wane Bridge against the flow lag time, and shows the tendency for short lag times to be associated with high flows and vice versa.

Graphs for Brockhill and Wane Bridge

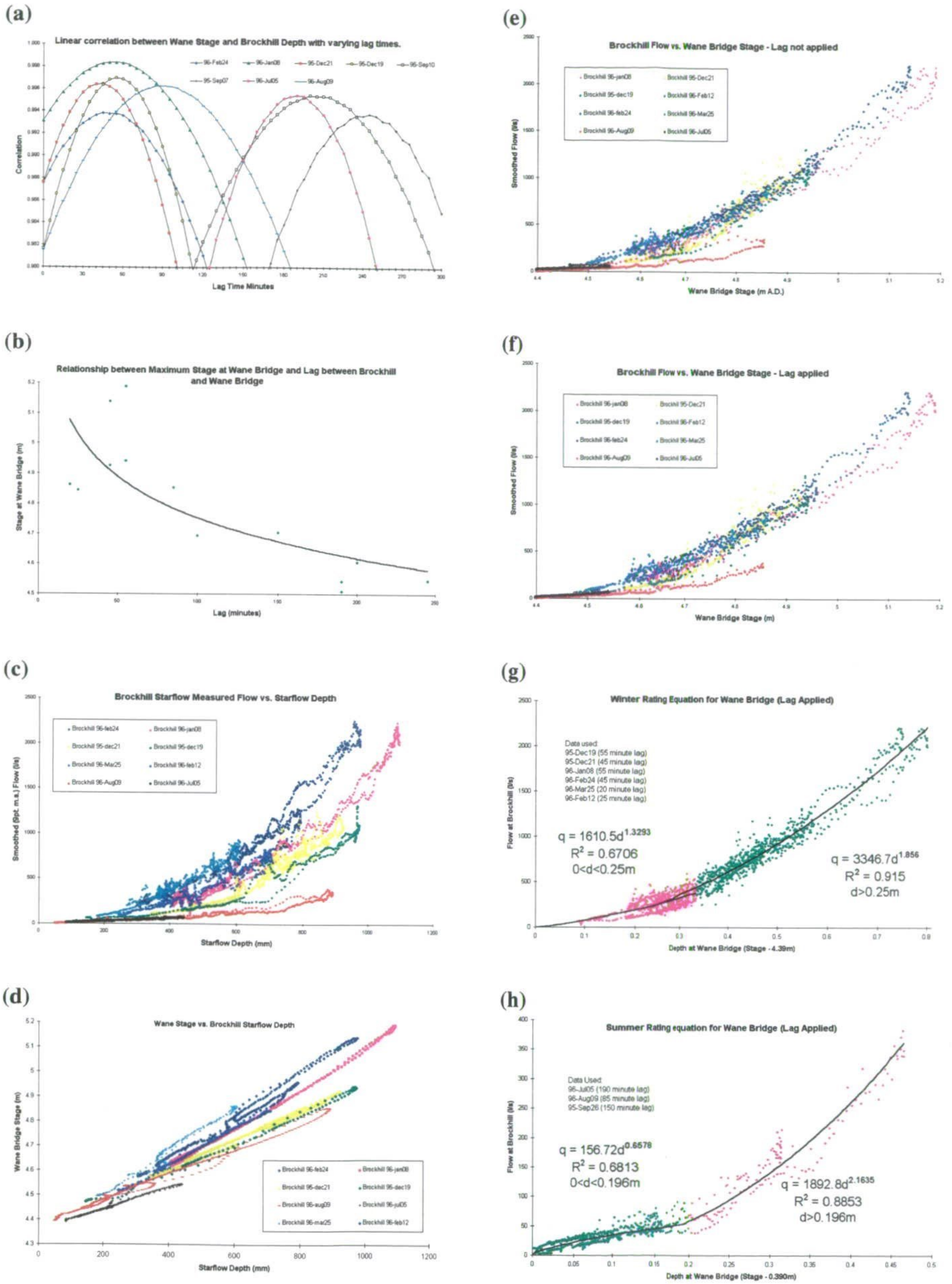


Figure B8

Table B6 Lag times on the Cut between Brockhill and Wane Bridge

Event Start Date	Lag (minutes) (max. correlation)	Brockhill Max. Depth (mm)	Wane Bridge Max. Stage (m)	Brockhill Maximum Flow (l/s)
24 July 1995	190	338	4.511	135.0
7 Sep 1995	245	430	4.547	172
26 Sep 1995	150	658	4.705	214.7
10 Sep 1995	200	252	4.608	317
6 Dec 1995	100	619	4.695	242.4
19 Dec 1995	55	972	4.943	1320
21 Dec 1995	45	923	4.927	1252
8 Jan 1996	55	1096	5.191	2239
12 Feb 1996	25	709	4.845	914.8
24 Feb 1996	45	975	5.141	2256
25 Mar 1996	20	609	4.864	935.2
5 July 1996	190	441	4.544	81
9 Aug 1996	85	890	4.855	396

Due to the rural nature of the river, higher flows were generally found in winter and lower in summer and it was difficult to isolate the effects of either season or flood magnitude for analysis. It is interesting to note the greater variability in lag time between Brockhill and Wane Bridge (a rural catchment) compared to that between Warfield House and Bull Brook Weir Site (a largely urban catchment) even though the distance between the latter two gauges is greater than that between the former. The urban gauges are also separated by Warfield House Pond. This discrepancy would appear to be due to the fact that the response of the urban catchment is similar throughout the year, whereas varying soil moisture content and changing channel vegetation characteristics cause the rural catchment to exhibit a more variable response over the course of a year.

B 3.4.3 Rating Equation

Firstly a flow rating was sought based only on the Brockhill data, plotting the Starflow discharge and depth data to see whether a consistent rating was observed. Unfortunately, as can be seen in Fig. B8c the rating for Brockhill Bridge changes a great deal over the year and for different storm magnitudes. This meant that a single rating equation could not be obtained for Brockhill and then applied to the levels at Wane Bridge to obtain flows for periods when the Starflow was not in operation.

Next, stages at Wane Bridge were plotted against depths at Brockhill. Data from each storm were offset against one another by a time equivalent to the lags given in Table B6. Again, the relationship throughout the year and for different flood peaks is highly variable, as demonstrated by the varying gradients obtained for each storm in Fig. B8d. However, it is evident that the trends exhibited in Figs B8c and B8d are similar: storms that produce a steep plot on the flow graph also yield a steep line on the stage plot. Thus it could be that the flow at Brockhill is related more consistently to the stage at Wane Bridge than it is to the depth at Brockhill (i.e. Wane is a more stable if less easily gauged section). By plotting flow at Brockhill against stage at Wane Bridge, this was found to be true. Figure B8e shows a more steady rating curve for Wane Bridge, even without lags being applied. Applying the relevant lag to each storm (Fig. B8f) has little visible effect on the appearance of the graph except perhaps to accentuate the fact that only the storms of 9 August 1996 and 5 July 1996 fail to fit the general trend. Applying lags has most effect on these two storms because the looping of the rating between two sites would be more prominent for intense summer storms than for winter storms where flow varies less rapidly due to the larger component of baseflow.

It was assumed that the two summer storms did not fit the general trend due to vegetation growth during the summer increasing the effective hydraulic roughness of the channel reach. It was thus decided to derive two rating equations, one for winter (no vegetation) and one for summer (with vegetation). By studying the additional storms in more detail, and by considering growing seasons and the time required for vegetation to die back, it was concluded that the 'summer' could be taken to last from June until September inclusive. This is of course a gross simplification, but it enabled a reasonable rating equation to be applied to almost all the storms, with larger uncertainties only occurring near the transitions between the seasons. The summer and winter storms were separated, relative lags applied and power laws fitted in two sections as in Figs B8g and B8h. The relevant rating equations are shown below.

Wane Winter Rating:	$Q = 1610.5d^{1.3293}$	$0 < d \le 0.25m$
	$Q = 3346.7d^{1.856}$	$d > 0.25m$
Wane Summer Rating:	$Q = 156.72d^{0.6578}$	$0 < d \le 0.196m$
	$Q = 1892.8d^{2.1635}$	$d > 0.196m$

where Q = flow discharge (l/s) and
d = depth(m) at Wane Bridge, given as stage - 4.39 m

B 3.5 Outfall at Jocks Lane

This outfall consists of three 1800 mm pipes (see Plate B3) discharging onto a short apron on the outside of a sharp bend in the Cut (see Fig. B9, marker F). Left to right (looking upstream) they apparently drain separate areas (Town Centre, Easthampstead, and Western Industrial area) but the pipes are cross connected allowing some 'leakage' to occur. Pipe flows were monitored separately at the first manhole (SU859703) upstream of the outfall, with the aim of combining them into one record covering approximately half of Bracknell's surface runoff to the Cut.

Flow was monitored under subcontract by ADS, using their depth/velocity monitors comprising downward looking ultrasonic depth gauges mounted in the soffit of the pipe and wide-beam ultrasonic/Doppler velocity meters mounted in the invert. A pressure transducer is used to measure depth under surcharge, but in free-flow the ultrasonic depth measurement is preferred. The data were telemetered to ADS offices, permitting direct problem identification and they were transferred to IH, at six to twelve month intervals.

The data were processed by ADS to provide depth H, velocity V and 'final' discharge Q, as well as a quality flag for each. Final discharge was normally derived by 'continuity' from the recorded velocity and depth (pipe geometry known). However, when velocity was suspect (flagged), flow could be estimated using a 'Manning' velocity as in equation B4 (start of this appendix). ADS use an exponent of 0.6 (instead of 2/3), and a 'fixed' conveyance (or 'hydraulic coefficient', as ADS call it) determined from separate manual readings of depth and velocity. By ignoring valid velocity measurements in adjacent periods, flow could change abruptly when changing to the 'Manning'

Gauge Locations: Jocks Lane (F) and Benbricke Green (H)

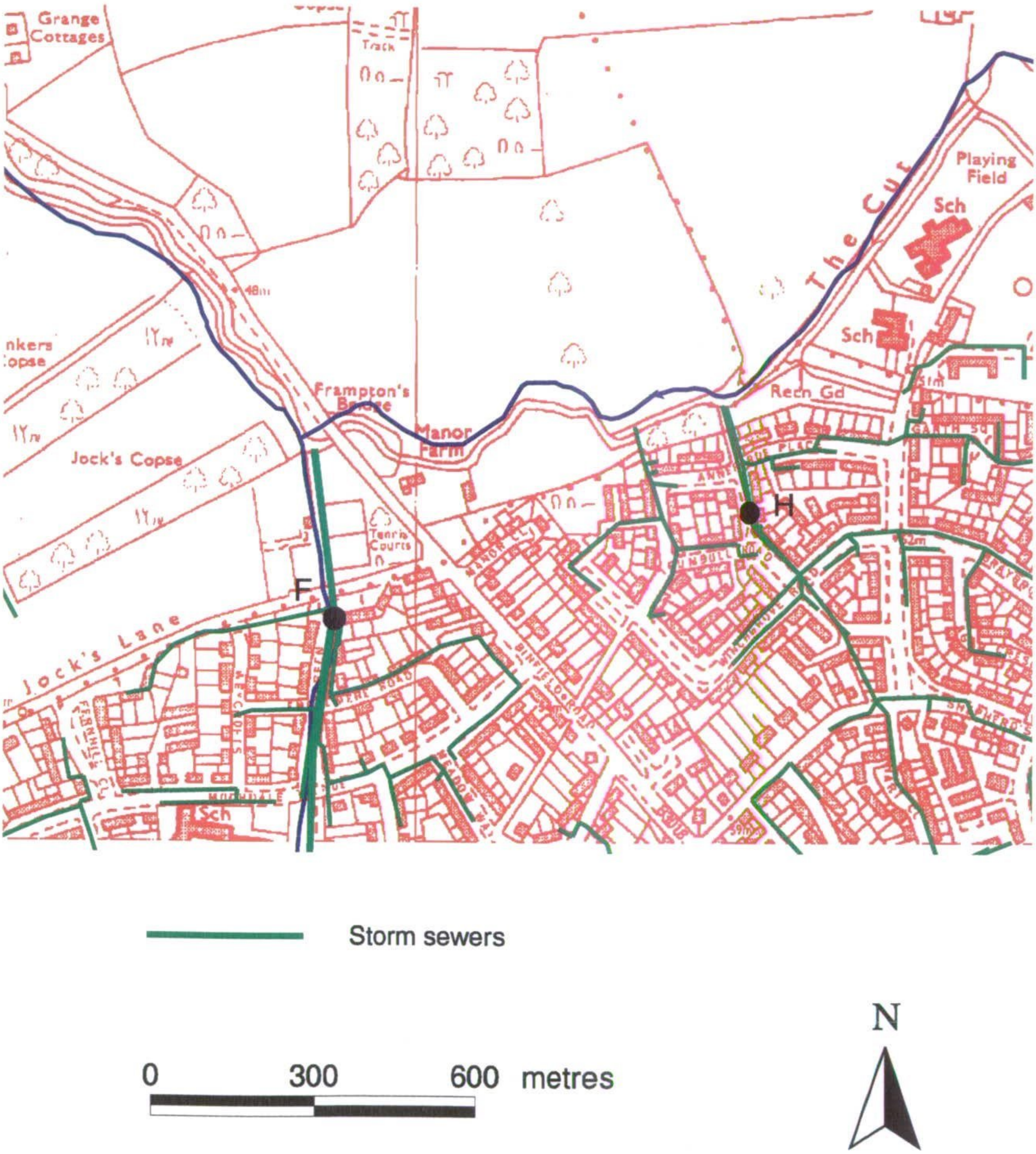


Figure B9

method. Also, the conveyance used varied surprisingly between the three pipes and over time (see Table B7).

Table B7 Default conveyance values used by ADS during data processing

Dates	Left pipe	Centre pipe	Right pipe
March 1993 to July 1993	3.39	3.444	2.336
July 1993 to March 1994	3.39	3.379	2.599
March 1994 to Oct 1994	3.39	3.379	3.028
Oct 1994 to Sept 1995	Not given	Not given	Not given
Sept 1995 to Aug 1996	2.093	2.228	1.591

These differences could be due to drift in hydraulic conditions or monitor set-up, or to observational error in the manual depth and velocity readings. The uncertainty was compounded as, despite the use of quality flags, it was not always clear when the Manning velocity had been used. As the impact of smoothing conveyance was being considered in this study, the ADS data was de-constructed in order:

- to define periods of missing, bad, or corrected data (where conveyance was used).
- to determine the conveyance used in comparison with periods of good data.

B 3.5.1 Time plots and scattergraphs

Figure B10 shows for each of the three gauges:

- a 'statusbar' (in black) of how the H, V and Q flags had been set by ADS.
- a trace (in blue) of conveyance C_Q (back-calculated from Q and D), and
- a trace (in red) of the ratio of conveyance C_V (back-calculated from V and D) to conveyance C_Q.

On the statusbar, a bad V flag plots under the bar, a bad Q flag plots as a short mark above, and bad H and Q flags plot as a long mark above (bad H should always give bad Q). Long periods of bad data plot as open 'boxes', while short periods produce spikes (which may merge into solid boxes). A long period of bad V is clearly seen for the left gauge in 1996-7.

These flags seem to be set by the ADS processing software (based on departure from some expected value), but the Q flags are normally cleared when Manning corrections are applied. Thus, if V is flagged but Q is not, Q has generally been derived by the Manning method (but sometimes the continuity method has been retained). If neither V nor Q are flagged, then Q has generally been derived by continuity. Sometimes Manning corrections have been applied when V is not flagged, and Q seems sometimes to have been derived by neither continuity nor Manning (see later). There are also times when V and Q are flagged but Manning corrections have not been applied, and times when Q is flagged but not V, and Q has not been evaluated at all. The Q flag is thus untrustworthy.

The statusbar is broadly reflected in the blue and red traces, with clear gaps where Q or V is zero and solid areas where there are sporadic zeros. The blue C_Q trace (plotted for C_Q<5) indicates the general variability of derived conveyance, becoming a horizontal line where Q has been derived from the Manning method (e.g. centre gauge in Apr 1993, left gauge in Jul-Aug 1995 and Feb 1996-Feb 1997 - short periods cannot be seen on this summary plot). The long period for the left gauge is of

Jocks Lane data checks

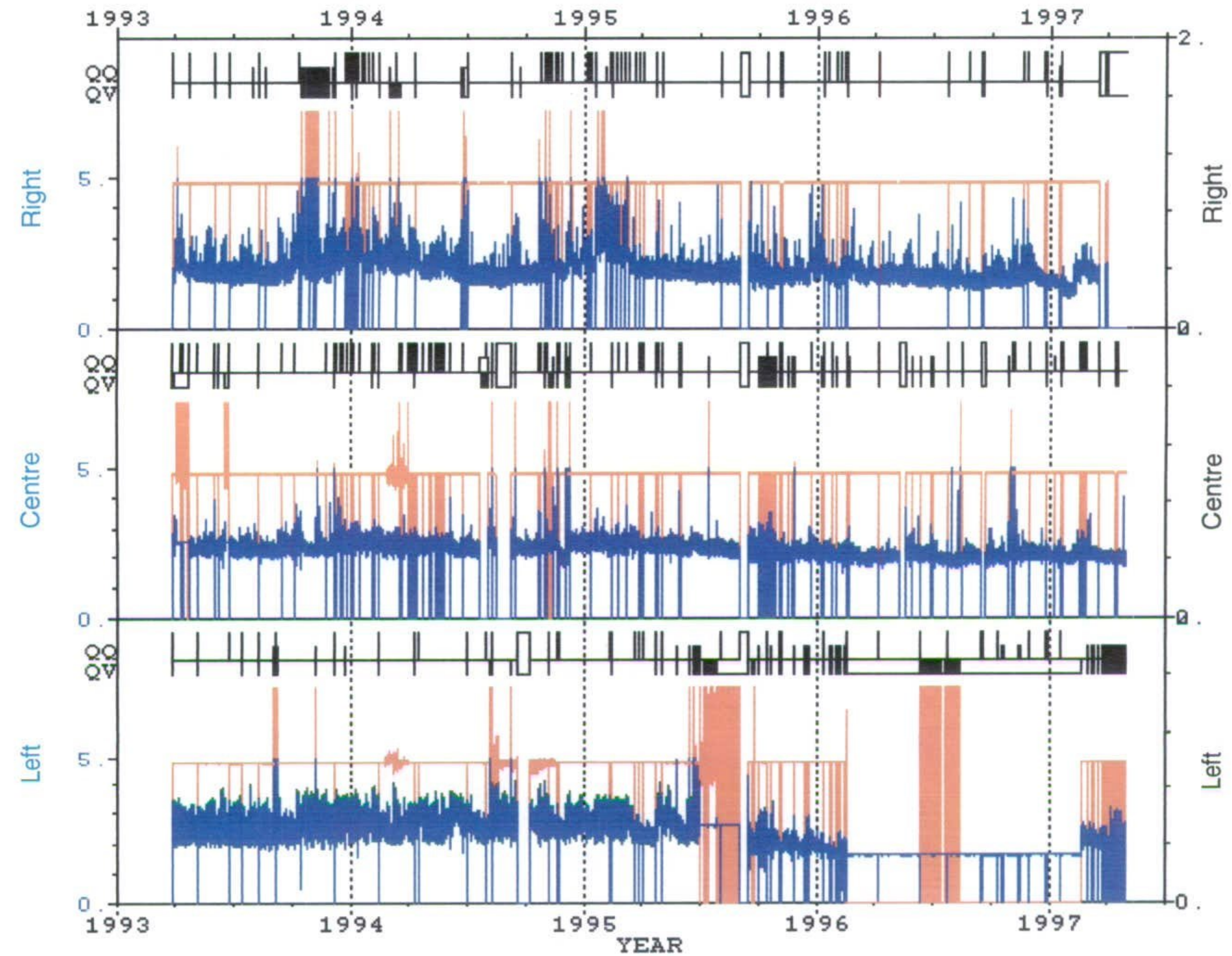


Figure B10

some concern as the C_Q value used by ADS is lower than would seem appropriate. The drift in the right trace is also of concern, but as will be seen is due to departures from the Normal-Depth/Manning equation for this pipe. The 'horizontal' C_Q values differ from the values in Table B7, which ADS explained as either a 'point-to-average' velocity or an 'imperial-to-metric' conversion. The blue traces do suggest a small change in conveyance has occurred, but not as large as the 35% reduction suggested in Table B7.

The red trace shows the ratio of C_V to C_Q. Where Q has been derived by 'continuity' this trace is close to unity (small deviations are probably due to differences in defining flow area from depth). However there are periods of quite large deviations (e.g. March 1994) when Q appears not to have been derived from continuity or Manning. Note also that for the left gauge the red area in mid 1996 relates to some 'good' V data giving C_V values comparable with the early record.

As a further check on the data, scatter diagrams of 'unflagged' velocity versus hydraulic mean depth were plotted for the whole record (Fig. B11). Here the black points (pre 1995) have been overlain by the blue (1995) and red (1996-7). The full range of scatter is large, but of approximately 400,000 points on each plot, most lie in the centre. The best fit lines of optimum and fixed slope (=0.6) confirm a slight reduction in conveyance over time (line 4 is the best overall fit). The circled points are derived from the manual readings made of depth and velocity. It may be noted that the right gauge shows a steeper relationship implying the Manning equation is not appropriate for this site.

B 3.5.2 Data reprocessing

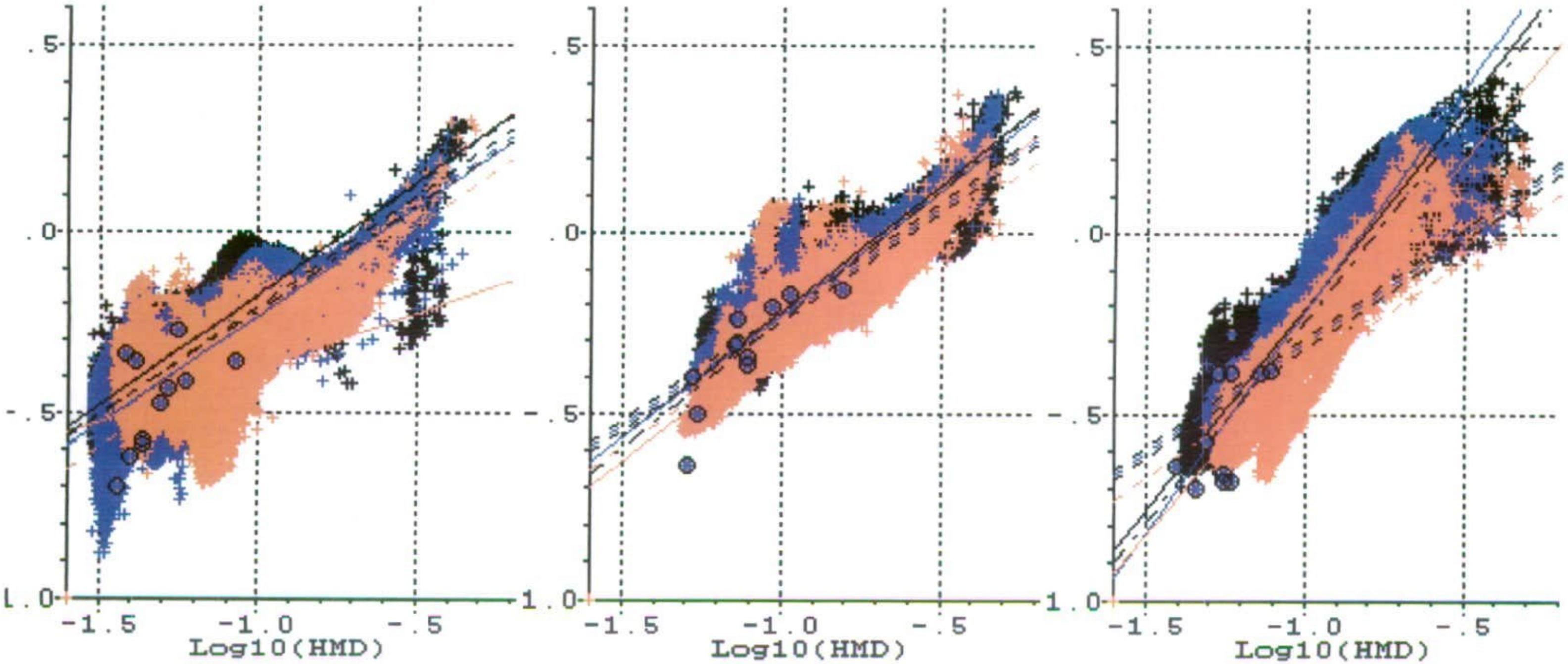
Based on the above analyses, it was decided that all the flow data needed reprocessing. All periods of missing or bad level data lasting more than 2 timesteps were examined to allow manual editing or infilling (shorter gaps were automatically filled by linear interpolation). A five-point moving average was applied to conveyance (derived from good depth and velocity) and long periods of bad velocity data were examined to confirm an appropriate 'carry over' value of conveyance was used in the Manning calculation of flow. Combining all three gauges, a near complete record of Jocks Lane flow (at 5-minute timestep) has been derived.

As well as the continuous record, 29 of the 31 selected event periods have also been extracted (2 periods in September 1995 were missing). For each period at each gauge (left, centre, right) a combined plot showing depth and velocity hydrographs and a scattergraph has been developed (e.g. Fig. B12). This plot shows the raw (blue) and the smoothed (black) velocity, and also the typical 'loop rating' effects in the scattergraph. The right gauge shows a rise in depth before a rise in velocity, while the left gauge shows a rise in velocity on the falling limb while depth continues to fall. These features were seen on almost every event and must be accepted as hydraulic realities, related (as at Oldbury) to changes between super and subcritical flow. They would seem to support a limited smoothing of conveyance rather than using a fixed value for gap filling.

B 3.6 Outfall at Benbricke Green (EA no. 2626/2627)

The Benbricke Green outfall, discharging at SU863704 (see Fig. B9, marker H), drains half the commercial centre of Bracknell, and the residential area to the North. It runs for the last 200 m as twin 1050 mm pipes. Detectronic 'sewer survey loggers', measuring depth by pressure transducer and velocity by Doppler shift, were installed in the second manhole upstream of the outfall. As with all the Detectronic loggers, data were collected at a basic 30-minute interval, switching to 5-minutes when depth exceeded 100 mm. Some hunting of the time interval occurred, and the manufacturer's processing software (SOFTDET/FLOAT) stored the data sequences separately for each interval. Software was written to merge the sequences at a fixed 5-minute interval, based on linear interpolation during the 30-minute interval periods.

Log (V): Log (Hydraulic Mean Depth) for gauges L, C and R



Line 1 V= 2.712*H** .607	Line 1 V= 2.969*H** .688	Line 1 V= 7.963*H**1.105
Line 1 V= 2.654*H** .600	Line 1 V= 2.404*H** .600	Line 1 V= 2.045*H** .600
Line 2 V= 2.277*H** .590	Line 2 V= 2.786*H** .673	Line 2 V=10.164*H**1.215
Line 2 V= 2.348*H** .600	Line 2 V= 2.333*H** .600	Line 2 V= 1.986*H** .600
Line 3 V= .840*H** .298	Line 3 V= 2.495*H** .685	Line 3 V= 5.225*H**1.029
Line 3 V= 2.034*H** .600	Line 3 V= 2.023*H** .600	Line 3 V= 1.677*H** .600
Line 4 V= 2.252*H** .568	Line 4 V= 2.928*H** .708	Line 4 V= 7.308*H**1.103
Line 4 V= 2.477*H** .600	Line 4 V= 2.254*H** .600	Line 4 V= 1.908*H** .600

Figure B11

Jocks Lane Event 09: Depth/velocity trace and loop rating

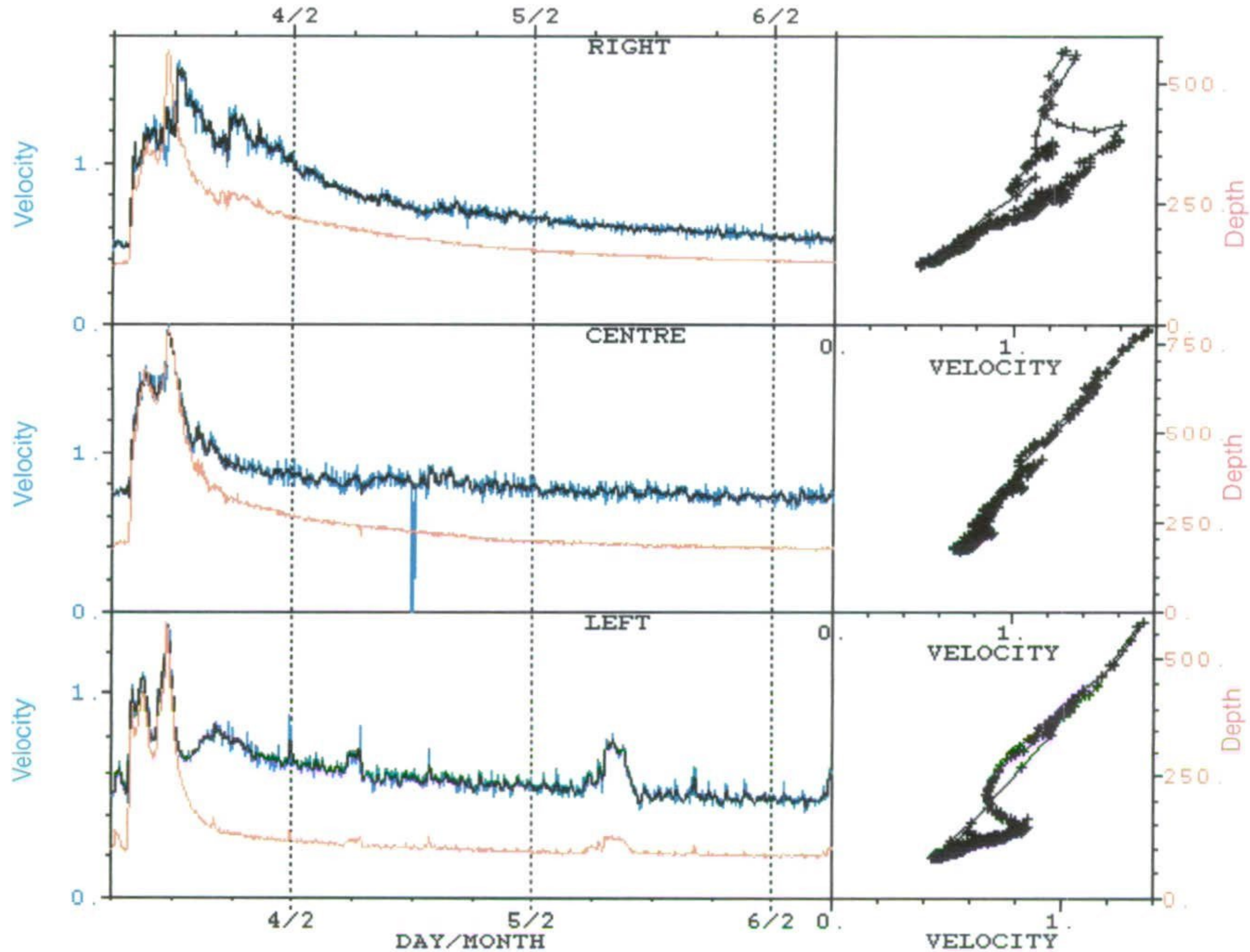


Figure B12

The data at Benbricke Green suffered from some problems. For example, the logging interval was too long to record accurately the rapid changes in depth and flow that could occur with 'unbalanced' runoff from an almost wholly urbanised area. Also, the velocity readings at Benbricke Green were found to be inconsistent/erratic, with, for example, periods of zero velocity occurring during storm events. As the fall from the monitor site to the Cut is quite large, and backing up should not occur, it was decided to use periods of reliable velocity data to derive a single rating applicable to both pipes. This would be applied to the depth data to create a more complete flow data series. Reliable data were identified by studying scattergraphs of depth and velocity data pairs.

Figures B13a and B13b show plots of flow against depth for the left and right pipes at Benbricke Green. Over 20 storms, spanning almost the whole period of data collection, were plotted for each pipe. Different colours indicate separate storm events. The ratings for left and right pipes are also plotted on a single chart (see Fig. B13c), which confirms that the two pipes have very similar ratings and that it is reasonable to derive a single rating equation serving both pipes.

The ratings were derived by fitting curves to the depth/flow data by eye (see Fig. B13d) and are as follows:

$$Q = 0.0004d^{2.5} \qquad d < 140mm$$
$$Q = 0.0013d^2 + 1.05d - 80 \qquad d \geq 140mm$$

where Q is the flow (l/s) and d is the depth of flow (mm)

These ratings were applied to all the depth data in place of the recorded velocity.

B 3.6.1 Filling in missing pipe depth data.

For the storms of 19 December 1995, 21 December 1995 and 8 January 1996, no data were available for the right pipe. Since the depths in the right pipe and left pipe are obviously closely linked it seemed feasible to estimate the right pipe values from the left pipe data. When data from the whole measurement period (1993-1996) were examined there seemed to be some seasonal differences in flow from the two pipes. The STC25 pipe layout data does shows that, at the pipe confluence that marks the start of the twin bore section, the catchment of the left hand incoming pipe includes some school playing fields. This might explain a preferential split of flow between the two pipes. In any case, the relationship between the two depths was difficult to define (see Fig. B13e), and it was decided that errors would be minimised by using the January 1995 storms to estimate right hand pipe depths for the following winter's storms.

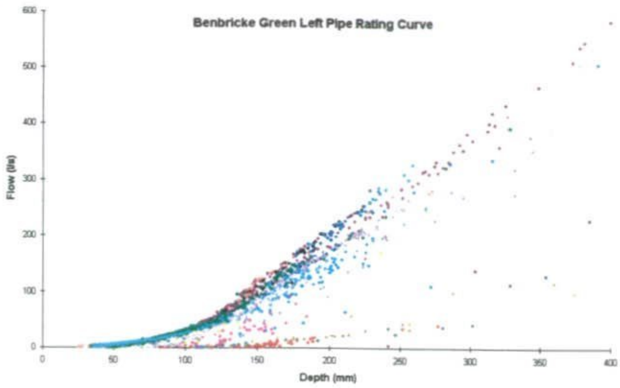
Depth data, selected from the rainy period between 17 January 1995 and 5 February 1995 were used to establish the relationship between the depth in the left pipe and that in the right (see Fig. B13f). By fitting a curve to the data, it was found that the depth in the right pipe was approximately related to that in the left by the formula

$$y = 3.2923x^{0.7202}$$

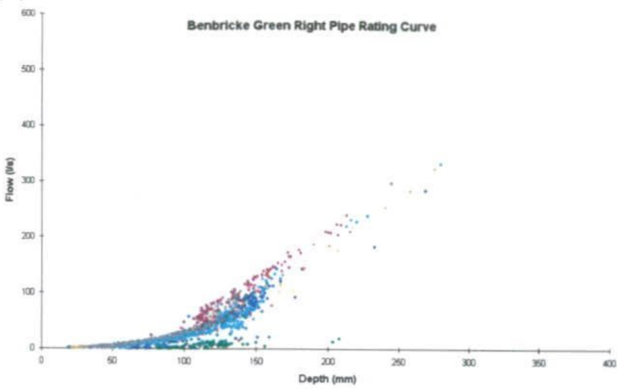
where x and y are the depths in mm in the left and right pipes respectively.
This relationship was considered sufficiently accurate for infilling data, given the known difficulties caused by depth changes too rapid to detect using a 5-minute logging interval.

Graphs for Benbricke Green

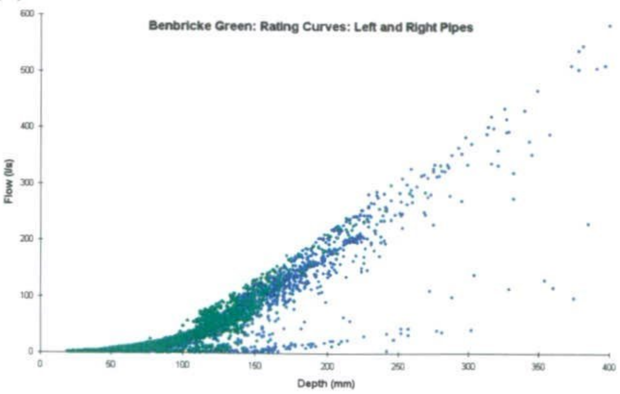
(a)



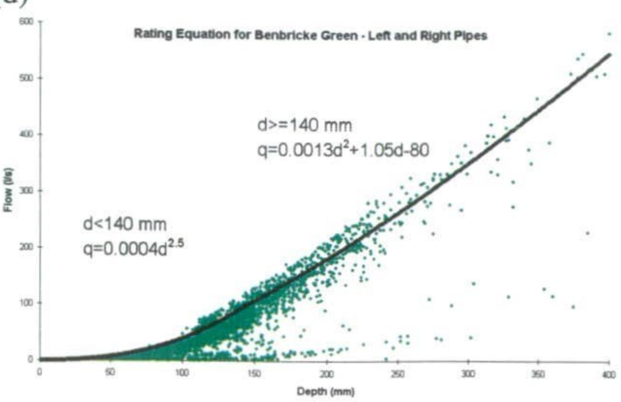
(b)



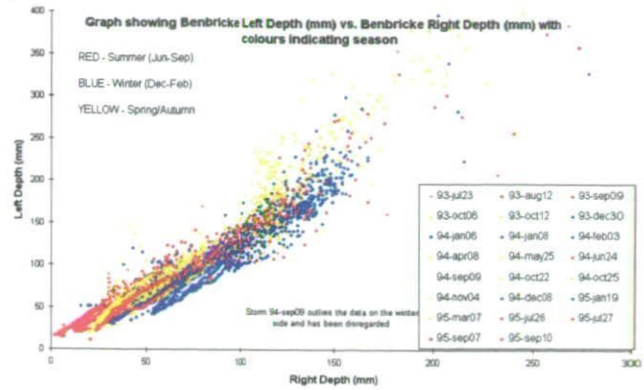
(c)



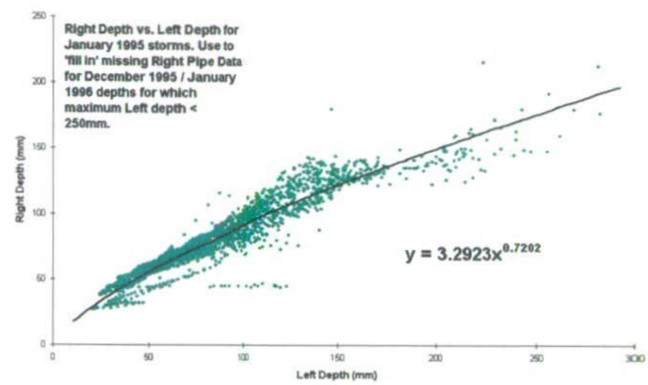
(d)



(e)



(f)



(g)

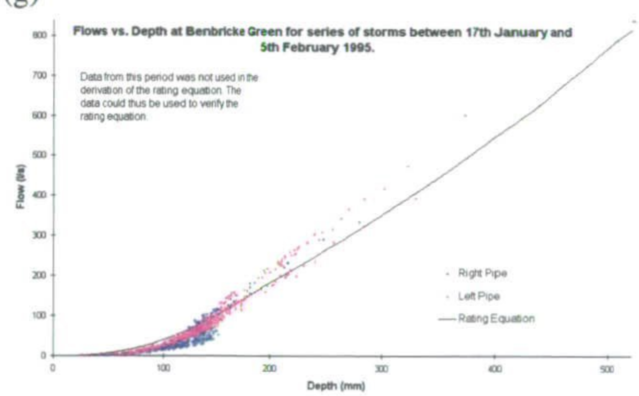


Figure B13

B 3.6.2 Verification of rating equation

Data from the period 17 January to 5 February 1995, which had not been used in the derivation of the rating equation, were used for verification. This data period contains the highest flow value in the whole record for either pipe. The applicability of the rating equation at high flows/depths was thus well tested (See Fig. B13g). The data and rating equation match well for low depths though perhaps less well above 200 mm. However, the data in this range is 'looped' and the equation does match well with one half of the loop. The equation predicts the flow at the highest depth (520 mm) remarkably well.

The rating equation was used for both pipes and all storms. The flows in the pipes were summed to give total Benbricke flows.

B 3.7 Mill Pond

This is a conventional on-line flood storage reservoir with the three inputs gauged by flumes installed (in the 1970s) in the inlet culverts. Plate B4 gives a view of the pond and shows the largest inlet culvert (at SU859679) draining the Easthampstead area (EA ref. no. 2605). This culvert is shown as '1' at B on Fig. B14, with the other inlets at Great Hollands (SU858680, EA No. 2603) and Wildridings (SU859683, EA No. 2604) shown as '2' and '3'. A further flume was installed in the outflow culvert '4', but a flow restriction downstream causes drowning at all stages. The original instrumentation involving chart recorders fell into disrepair, and the records are of very little use to this project. However, new pressure transducers were installed in the inlet stilling wells in June 1993, and data collection was restarted using a 15-minute timestep. The instrument gauging the Wildridings catchment was vandalised in early summer 1995 and not replaced until September 1996. The gauges for Great Hollands and Easthampstead provided a more complete data record with the former inactive only for the latter half of 1995 and the latter inactive from December 1993 to March 1994. The outlet flume could have been gauged using a sewer monitor, but at some inconvenience; the culvert is quite deep and access would require extensive safety procedures. However, the outlet culvert passes downstream, without addition for 870 metres, to outfall at the confluence with the controlled outlet from the Oldbury pond (see below, Section B 3.8). A sewer flow monitor (Detectronic: EA No. 2623) was installed at this point in June 1993, measuring flow at 5-minute intervals. Three different instruments were used over the monitoring period to measure flow for this outfall. The first was in operation from July1993 to April 1994. The velocity sensor then began to fail and the instrument was replaced with another Detectronic from April 1994 to December 1995. No flow data are available for this site between January and October 1996, but a third instrument (Prolec Water-Rat) was in operation from November 1996 to March 1997.

By considering the theoretical hydraulic conditions created by the flumes, rating equations were derived and level data were converted to flows for the storms studied. The equations employed for each Mill Pond inlet are shown in Table B8.

Gauge Locations: Mill Pond and Oldbury Pond

Storm sewers

0 200 400 metres

N

Figure B14

Table B8 Rating Equations for Mill Pond inlets.

Mill Pond Inlet	Rating Equation (Q in m^3/s)	Valid Depth Range (m)
Easthampstead	$Q = 4151d^{3.58}$	$d<0.02$
	$Q = 41.9d^{2.41}$	$0.02<d<0.032$
	$Q = 6.06d^{1.85}$	$0.032<d<2.1$
Great Hollands	$Q = 738d^{3.22}$	$0<d<0.02m$
	$Q = 18d^{2.28}$	$0.02m<d<0.039m$
	$Q = 3.24d^{1.755}$	$0.039m<d<0.153m$
	$Q = 2.168d^{1.54}$	$0.153m<d<2.1m$
Wildridings	$Q = 73.9d^{2.53}$	$0<d<0.01m$
	$Q = 5.13d^{1.94}$	$0.01m<d<0.033m$
	$Q = 1.72d^{1.62}$	$0.033m<d<0.104m$
	$Q = 1.385d^{1.52}$	$0.104m<d<1.2m$

Upon analysis of the data it became clear that the 15-minute interval was not sufficiently short to effectively measure the flow profile during a storm. The three areas all drained very rapidly due to their high amount of impervious surfaces coupled with relatively steep pipe and land slopes. In October 1996 the interval was decreased to 5 minutes in an attempt to record some storm events in more detail Figure B15a shows the storm responses of the inlets and outlet to Mill Pond for the storm of 23 July 1993. The graph shows how Easthampstead is the dominant inflow to the pond (catchment area = 517 ha) and it demonstrates the expected minor effect of the Wildridings catchment (catchment area = 14 ha). Inflow from the Great Hollands is approximately in proportion to Easthampstead, given the catchment area of 229 ha. The lag of the peaks between the Mill Pond Outlet flows and total inflows is 25 minutes for the first peak and 20 minutes for the two subsequent peaks. However, the inlet monitors were recording at 15-minute intervals over this period so these data only give lag time approximately. The graph also shows that far more water appears to enter the Mill Pond during some storm events than is *recorded* leaving it. This indicates a problem with the equipment or rating equations.

By investigating the depth and flow recorded at the Mill Pond Outlet (at Oldbury) for a number of different storms, the performance of the logger and flow conditions were evaluated. Figures B15b-d show the highly variable ratings obtained over 3 years of data collection. The ratings are not changing slowly and regularly over time (as might be expected were logger drift to blame) but they vary within and between storms. Each of the three time periods contains storms which match and lie on the steepest (maximum) rating obtained. All periods also contain storms where the rating is much lower. This suggests that the logger is capable of recording flow consistently but that perhaps some channel obstruction or debris on the sensor is causing under recording of velocity for some storms. Surprisingly all ratings demonstrate a 'normal depth' form of relationship between depth and flow.

B 3.7.1 Volume balance for Mill Pond

To confirm whether or not the lower ratings were indicative of true flow, the volume balance (after removing 'baseflow' from each record) through the Mill Pond was investigated. Since all the instruments involved were working simultaneously for only 5 out of 37 selected storm events, a volume relationship was needed between the inflow catchments so that the total inflow could be estimated even when only 1 or 2 of the input depth recorders were operational.

Graphs for Mill Pond

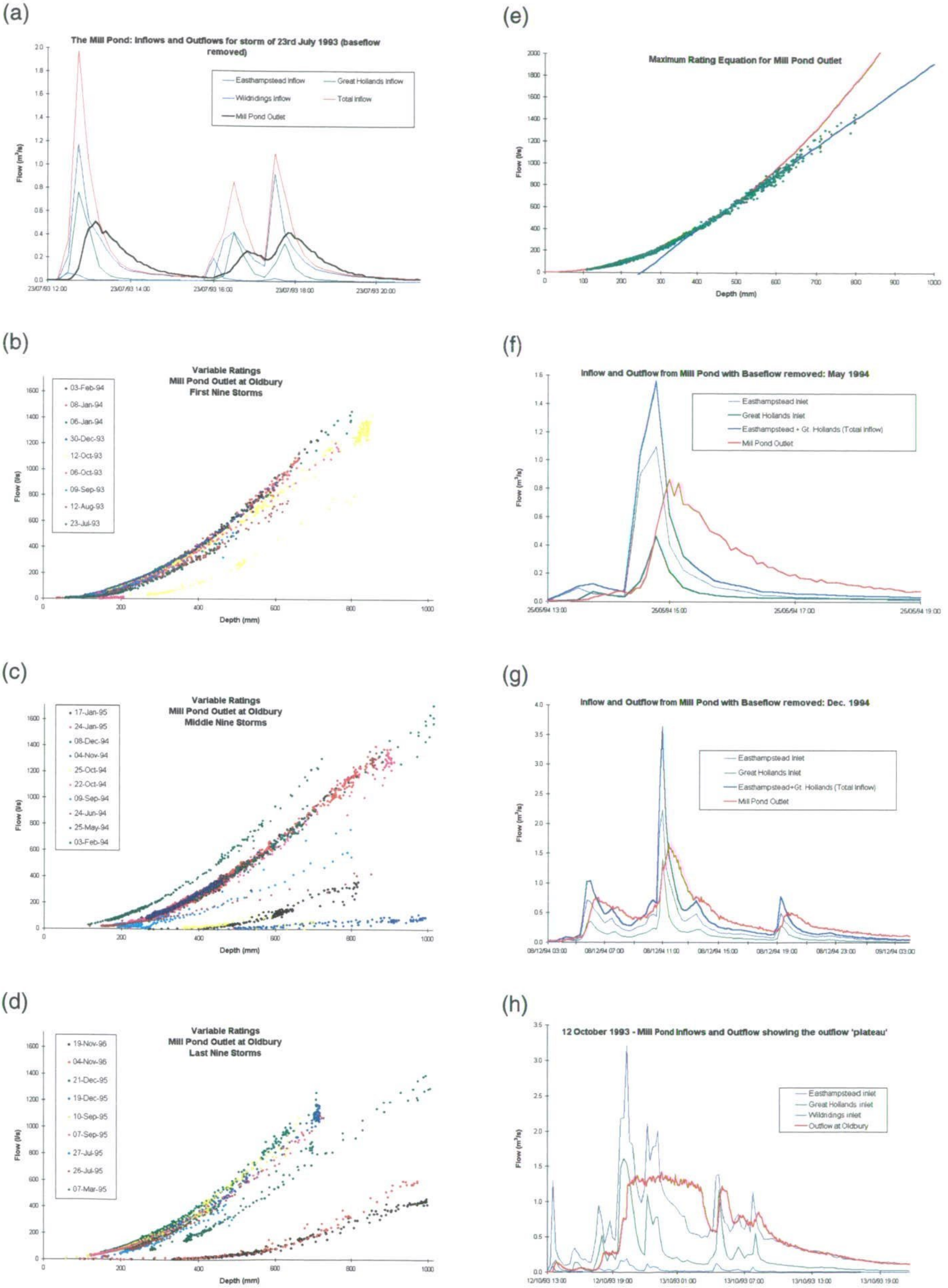


Figure B15

Volumes measured for Great Hollands and Wildridings catchments were compared with those for Easthampstead (See Table B9). The table also gives the average percentage volumes which were used to estimate inflow volume at a gauge when it was not functioning.

Table B9 Inflow volume comparison for Mill Pond

Catchment Name	No. of storms used in calculations	Average % of Easthampstead Volume	% of Easthampstead Catchment Area	Maximum % Recorded	Minimum % Recorded
Great Hollands	16	42.1	44.3	53.4	35.8
Wildridings	7	5.2	2.7	6.7	3.8

n.b. The maximum percentage recorded for Great Hollands was 87.8%, for the storm of 24 June 1994. This value was disregarded since it appears that the 15-minute interval meant that the Easthampstead instrument completely failed to record the peak discharge associated with that storm.

The total volume estimated to have entered the lake for each storm event was then compared with that measured leaving the lake. If all the data were accurate then it would be expected that the two volumes would almost match, with slightly more water leaving the lake due to rainfall landing directly on the water surface or draining off its banks. The values for each storm are shown in Table B10.

Table B10 Comparison of measured outflow and inflow volumes for Mill Pond

Storm Date	Outflow/Inflow %	Storm Date	Outflow/Inflow %	Storm Date	Outflow/Inflow %
19 Nov 1996	16	17 Jan 1995	142	3 Feb 1994	58
4 Nov 1996	23	24 Jan 1995	150	8 Jan 1994	60
21 Dec 1995	80	8 Dec 1994	102	6 Jan 1994	63
19 Dec 1995	73	4 Nov 1994	4	30 Dec 1993	62
10 Sep 1995	98	25 Oct 1994	12	12 Oct 1993	65
7 Sep 1995	87	22 Oct 1994	90	6 Oct 1993	71
27 July 1995	84	9 Sep 1994	58	9 Sep 1993	62
26 July 1995	117	24 Jun 1994	57	12 Aug 1993	58
7 Mar 1995	80	25 May 1994	110	23 Jul 1993	55

Table B10 clearly shows that the flows measured leaving the pond are poorly estimated for many of the events, as already indicated by the varying rating curves seen in Figs B15b-d. The most stable percentages, of outflow to inflow, occur for the 9 storms monitored before April 1994 (i.e., with the first flow meter installed) and this period corresponds to the most stable rating for Mill Pond Outlet. The values closest to 100% were obtained from data collected by the second logger though, due to the high variability over this period, these results seem spurious with much higher percentages occurring for long duration events than for short, suggesting that measurement during low flow was inaccurate. For several storms, very low outflows were also recorded suggesting that the second and third instruments were less reliable than the first. Overall, it seems that the steepest rating curves obtained are most likely to represent actual flow conditions due to this rating being obtained consistently for the first logger and for part of the period of operation of the second logger. This rating does not provide the best volume balances, but this is likely to be due to inaccuracies in the theoretical rating equations for the three Mill Pond inlets, where only depth data are available. The 15-minute time step is also clearly a contributing factor. Taking the maximum ratings as the most appropriate, the preferred rating equations for Mill Pond outlet are:

$$Q = 0.0013d^{2.11} \qquad d \le 0.450m$$

$$Q = 2.55d - 633.5 \qquad d > 0.450m$$

where Q = flow (l/s), d = water depth in the pipe (mm)

The rating is shown in Fig. B15c. However, it may be noted that the Mill Pond data used to compare with modelling results in Chapter 5 of the main report were based only on monitored velocity data.

B 3.7.2 Effect of Mill Pond on the Hydrograph

In order to analyse the effect of Mill Pond on the flood peaks, only events where the inflow and outflow volumes are approximately equal were considered. The storms of 8 Dec 1994 and 24 May 1994 had closely matching inflow and outflow volumes and the hydrographs for these storms are shown in Figs B15f and B15g. Unfortunately, data for Wildridings is not available for either storm though this would have had little effect on peak inflows since the very small catchment yields far lower peak flows than Easthampstead (<10%). Also the peak inflows from Wildridings reach the Mill Pond before the peak flow from either the Easthampstead or Great Hollands catchments. The peak total inflow for the December storm was 3.63 m^3/s and the peak outflow was 1.65 m^3/s, just 45% of the peak inflow. The lag between these peaks was 30 minutes. For the May storm, the peak outflow lagged the inflow by 15 minutes and was 55% of the inflow (0.86 m^3/s compared to 1.56 m^3/s). Again, the long logging interval of the inlet monitors mean that timings are approximate and peak flows may have been missed. However these values suggest that the Mill Pond attenuates flood peaks by approximately 50% and delays them by somewhere between 15 and 30 minutes. This is the situation for the small and medium events but for large events, the lake outlet can act as a throttle, as discussed below.

B 3.7.3 Throttle effect of Mill Pond outlet pipe

For certain large storm events, the depth at Mill Pond Outlet seems to reach a maximum depth (though not pipe full depth) and records this 'plateau' depth (or very close to it) for an extended period. This did not happen for any other gauge in the study. Figure B15h shows the storm of 12 October 1993 when depths of between 801 mm and 856 mm were recorded for almost seven hours. Corresponding to this depth, the flow rate was about 1300 l/s. This plateau may be due to the outlet pipes (two 685 mm diameter pipes) from the Mill Pond outlet-weir chambers acting as throttles when the lake level rises, resulting in near constant high flows at the downstream outlet. Theoretical ratings for these pipes indicate they would 'prime' at a discharge of about 1.8 m^3/s, suggesting some additional throttling may occur within the culvert. The STC25 sewer data show the culvert as 1300 mm for most of its length, but this changes to twin 900 mm pipes for one road crossing, with a backdrop indicated. Further work would be needed to determine the full cause of the plateau effect.

The average 'plateau' depths recorded vary from 995 mm down to 722 mm (See Table B11), but maximum flow is consistent for the first 3 storms which were all within 2% of each other.

Of the remaining two events, the November 1996 storm only achieves a small maximum flow and the instrument was known to be under recording velocity. The December 1995 storm falls short of the first three by 15% both for depth and flow, suggesting that differences are due to depths being measured incorrectly.

Table B11 'Plateau' flows at Mill Pond outlet

	Average Depth (mm)	Minimum Depth (mm)	Maximum Depth (mm)	Average Flow (l/s)	Duration (hr:min)	Logger
12 Oct 1993	832	801	856	1296	6:40	Detect. # 1
24 June 1994	859	851	868	1276	1:50	Detect. #2
22 Oct 1994	900	891	914	1289	1:50	Detect. #2
19 Dec 1995	722	702	718	1094	3:45	Detect. #2
19 Nov 1996	995	990	999	416	0:50	Water-Rat

For the first three storms it was considered likely that peak outflows were measured correctly. Thus, the maximum lake inflows and outflows could be compared to ascertain flood peak attenuation and lag (see Table B12). The peak outflow is taken as the average flow at maximum (plateau) depth.

Table B12 Attenuation and lag times when throttle effect occurs at Mill Pond

	Maximum Inflow (m^3/s)	Maximum Outflow (m^3/s)	Percentage Attenuation	Lag Time (minutes)
12 Oct. 1993	4.88	1.30	73%	30
24 June 1994	6.19	1.28	79%	10
22 Oct. 1994	4.53	1.29	72%	5

Note The lag time estimates are subjective due to the lack of a sharp peak in the outflow hydrographs and the 15 minute logging interval at the inlets.

These results demonstrate how, for the larger storms, the effect of the Mill Pond on flood peaks is considerably increased due to throttling at the Outlet or within the culvert.

B 3.8 Oldbury pond

This 'pond' comprises a channel with side weir overflow to an off-line 'dry' storage area. The pond has been considerably altered since it was built. Originally used as a horse paddock, it was first extended and then re-developed as a wasteland under a 'car-park on stilts'. Views of the weir, channel and pond are given in Plate B5. The two inlets (at SU857689) are both 1300 mm pipes, the Eastern pipe (marker '1' at G on Fig. B14) draining a mixed residential and commercial area, and the Western pipe (marker 2) draining the 'Waitrose' estate. The channel outlet (or side weir 'Bypass') is a single 1050 mm throttle pipe which outfalls some 80 m downstream (marker '3') alongside the outfall culvert (marker '4') from the Mill Pond (see Plate B6a with the Oldbury bypass outlet shown beyond the Mill Pond outlet). The combined flow then passes in culvert (Plate B6b) under the motorway link road and railway line. The pond has a small direct inflow from the link road (briefly monitored in 1993). It drains via two 200 mm pipes passing separately under the road and railway before linking back to the main culvert. An emergency overflow weir discharges over the steps between the 'Bypass' and 'Mill Pond' outlets (Plate B6a).

This study has concentrated on the behaviour of the side weir channel, and flows out of the pond have not been gauged. Detectronic sewer monitors were installed in June 1993 on the 'Industrial' inlet (EA No. 2625), the 'Waitrose' inlet (EA No. 2624) and at the downstream end of the 'Bypass' throttle pipe (EA No. 2622). As at Benbricke Green, data were logged at 30-minute intervals for flow depths below 100 mm, and at 5-minute intervals for higher depths. Storm periods have been extracted from the recorded data and viewed as scattergraphs. A generally high degree of variability was found, including 'loop rating' effects due presumably to the differences between water surface and pipe slopes over rapidly rising and falling depths. Quite pronounced loops were found for the two inlets which were affected by backing up from the downstream throttle.

Figure B16a shows a typical loop rating for the Industrial inlet. Initially, from A to B, velocity varies linearly with depth, but between B and C velocity increases more rapidly. This could be due to

- the velocity sensor having been cleared of some initial obstruction,
- a larger depth over the sensor with less relative disturbance to the flow pattern,
- changing hydraulic conditions, or
- the effect of a steeply rising flow profile, which from hydraulic considerations would predict such an anti-clockwise loop.

Up to point C, the water depth continues to increase until the depth of water in the weir channel backs up in the inlet pipe and from C to D slows the flow. At this stage, a stratified flow profile could be present, with higher velocities near the water surface causing poor measurement of average velocity. As the storm recedes the depth drops while the velocity remains steady (weir channel draining). As the water level drops further, backing up ceases and velocity increases. After the main storm the depth/velocity relationship reverts at E to a (different) linear profile.

This basic trend in velocity measurements was repeated for a number of storms, and its smooth progression seemed to represent true flow patterns rather than erroneous readings (unfortunately the channel has never been properly observed in a storm event). As the loop effect easily outweighed other possible problems (such as noise in the velocity sensing, or drift in depth sensing), little or no smoothing was applied to the data, except where data values were missing. Specific issues relating to individual monitor locations are discussed in the following sections.

B 3.8.1 Industrial Scattergraphs

The scattergraphs for individual events at the Industrial Inlet sometimes indicated a good measurement of the flow conditions, with velocity following a near linear relationship with depth at low depths, but diminishing at higher depths due to backwater from the weir channel. However, when the scattergraphs from several storms were compared (see Fig. B16b) it was clear that the relationship between the depth and velocity was unsteady. Similar depths corresponded to velocities differing by approximately 0.5 m/s and for certain storms the maximum velocity was less than 0.5 m/s, far lower than would usually be expected for the

Graphs for Oldbury Pond, Jealotts Ditch and Worldsend

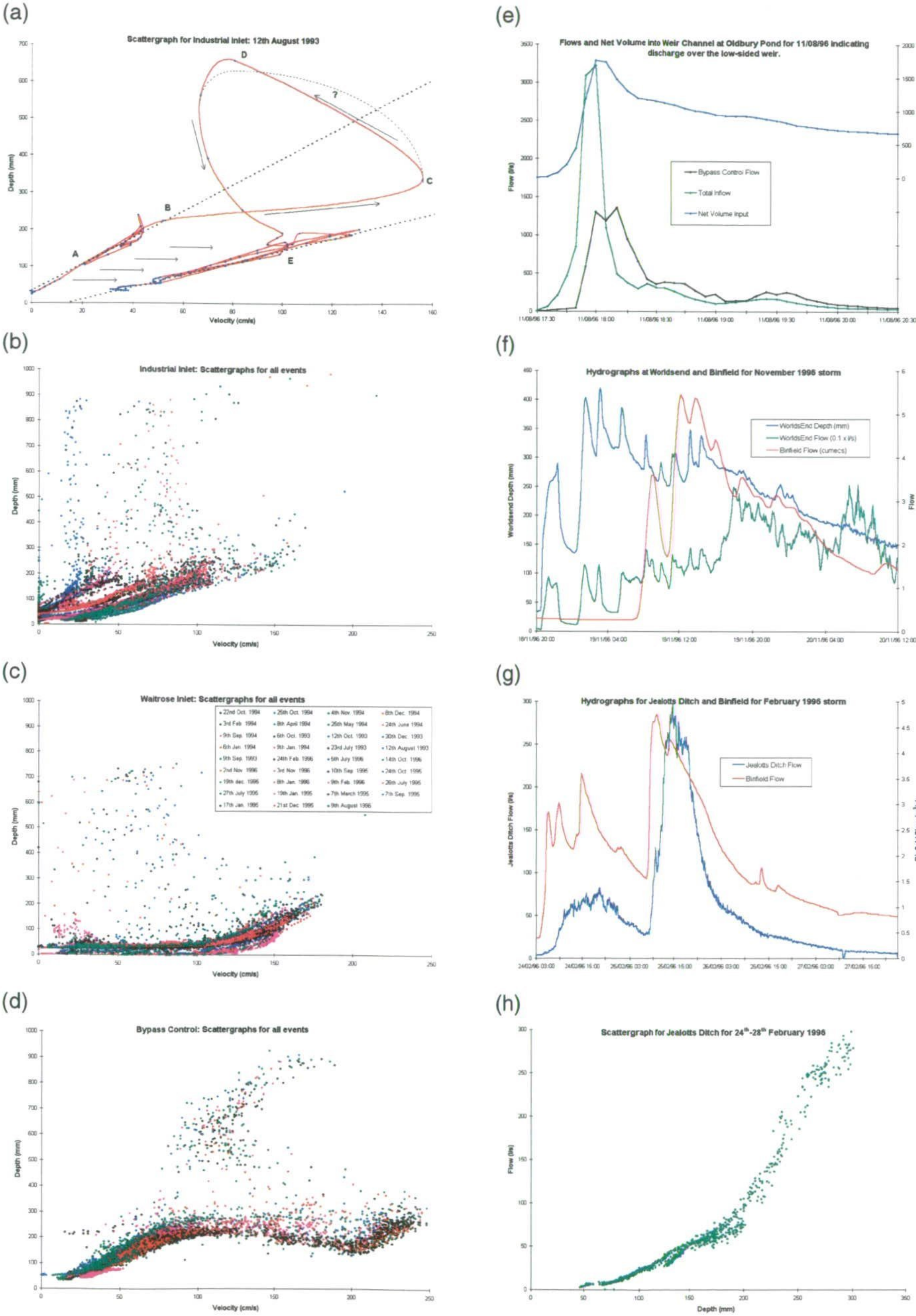


Figure B16

depths achieved. These shifts in the rating proved difficult to correct due to the haphazard nature of the shifts and therefore the raw data were used in the calculation of flows.

B 3.8.2 Waitrose Scattergraphs

The scattergraphs for the Waitrose Inlet are generally consistent (see Fig. B16c) though not indicative of good quality data. During the rising limb of a storm event, velocity rises to approximately 1m/s before a corresponding increase in depth is recorded (the inlet pipe is steep and the flow is supercritical). In the Industrial Inlet or Bypass Outlet a similar velocity was recorded for depths of approximately 200 mm yet here it is regularly obtained at depths of under 50 mm. Under these conditions, drift in depth measurement, flow disruption by the sensor, and problems of making manual check measurements in shallow streaking flow make accurate flow monitoring virtually impossible. It is probable that the depth is under recording, but corrections have proved too complex to assess with confidence, even based on cross correlation with the Industrial Inlet. Again, the raw data have been used in the calculation of flows.

Note that prior to 27 September 1994, flows were calculated using a depth of flow 25 mm greater than that recorded. This was due to a manual measurement and readjustment of the logger offset. The scattergraphs show the depth data without this 25 mm adjustment.

B 3.8.3 Bypass Scattergraphs

Of the three gauges, the Bypass/Outlet is the most consistent, and the scattergraphs of Fig. B16d all indicate the same flow pattern. Velocity increases almost linearly with depth for depths below about 200 mm. Then velocity grows (from less than 1 m/s to greater than 2 m/s) while depth seems to drop slightly. Reaching a maximum velocity of about 2.4 m/s, depth starts to rise again to about 500 mm as velocity falls to about 1 m/s. Finally velocity starts to increase again up to about 1.5 m/s at a depth of about 900 mm. The likely explanation is that:

- critical flow occurs at the inlet to the throttle pipe
- at low flows a hydraulic jump occurs in the throttle pipe and the flow at the monitor location downstream is subcritical,
- at mid flows the flow remains critical right through to the outfall, and
- at higher flow, increased pipe resistance (or maybe downstream conditions) force the transition back into the pipe.

In any case, despite the high recorded velocities, the data are the most reliable of the three Oldbury gauges, and have been used raw in the calculation of flow.

B 3.8.4 Comparison of the three gauges

Data from each gauge viewed separately seemed to be of a reasonable standard, but comparing the gauges identified a number of discrepancies. Combining the derived inflows from the Waitrose and Industrial monitors almost always gave lower discharge than recorded by the Bypass monitor, both over the duration of an event and at instantaneous times in low flow conditions. This difference was clearly not due to any influx between the measuring points but could have been due to velocity monitoring problems at the very low depths in the inlet pipes. Depth hydrographs in the inlet pipes generally matched the shapes at the Bypass, but the velocities were quite different. Inaccurate velocity monitoring at low depth may be due to insufficient cover over the Doppler sensor, or to the 'mouse' impeding the flow. The combined flow in the (smaller) outlet pipe ran at a greater depth, and the instrument may have recorded velocity more realistically.

Although inlet volume data was untrustworthy, the data could be used for verifying routing times and examining the effect of the weir channel on flows.

B 3.8.5 Operation of the Low Sided Weir

The lack of consistency between the three Oldbury gauges has made it difficult to determine when the water level in the weir channel was sufficient to overtop the weir. However, in the summer of 1996, some scour was observed on the dry side of the weir adjacent to the throttle entrance, and grass was observed flattened and pointing away from the weir, indicating that the weir had operated (at one end) at least once. Of all the monitored storms, only two showed greater maximum inflow than outflow, on 24 June 1994 and 9 August 1996. Unfortunately, the flow values recorded for Waitrose in June 1994 fail to reach high values but for the August 1996 storm, the maximum flow for the whole data period was achieved for both the Industrial (2.108 m^3/s) and Waitrose (1.397 m^3/s) Inlets. The maximum flow for the Bypass Control (1.448 m^3/s) was within 0.05 m^3/s of the other six highest Bypass flows, suggesting that these flows represent the maximum for the outlet pipe (given the head defined by the side weir). Figure B16e shows the total inflow (green) and outflow (black) hydrographs, with the cumulative difference (in blue, above) showing the net input into the weir channel. This suggests that approximately 600 m^3 of water entered the channel but did not leave by the Bypass Control and therefore must have left via the side weir into the off-line storage area. According to the pond area data of section B 1.2.13, this would equate to about 0.5m of water at the pond outlet.

The figure of 600 m^3 is however error prone, given the known problems with the gauges and the long logging interval (5 minutes) for such a rapid response. A broad check on this figure can be made by noting that the physical volume of the weir channel up to the side weir should equate to the difference between inflow and outflow volumes (i) prior to side weir operation, and also (ii) from when weir flow ceases on the recession. The calculation is made more complicated by the flow time from the throttle inlet to the flow monitor. However, incorporating a one interval offset (as suggested by Fig. B16e), and making some assumptions as to when in the timestep side weir flow begins and ends, volumes (i) and (ii) were estimated as approximately 270 m^3. This compares quite favourably with the weir channel volume (65 m long, 3 m wide and 1.05 m deep on average) of 205 m^3.

B 3.9 Worldsend (SU886671)

This gauge monitors a culvert draining from a forest catchment (area 2.04 km^2) under the 'Forest Drive' road that bounds Bracknell urban development (see Fig. B4, marker J). Plate B7 shows the forest stream(a) and the culvert inlet(b), while views of the forest and 'Drive' in the vicinity can be seen in Plate B11. This culvert inlet seems totally blocked by forest detritus (the lip of the concrete pipe can just be seen), but this natural 'baffle' is fairly open and large flows can pass through (see Plate B7c). Downstream there is a confluence with a small urban storm channel (see Plate B7d), which continues, partly open and partly in culvert, to the Savernake storage pond. Initially it was intended to install a weir upstream of the culvert, but problems of construction, vandalism, and keeping the approach free of detritus were too great. Instead, a Prolec Water-Rat monitor was installed about 5 metres up from the culvert outlet (Plate B7c) in February 1995. Although well concealed and protected, the instrument was vandalised in June. It was repaired and replaced in October, but failed in April 1996. It was finally replaced with a Unidata Starflow from August 1996 to March 1997. Data have been collected at a 5-minute logging interval.

Initial analysis of the data showed a lot of 'noise', not unexpected for measuring generally small flows in a pipe. By replacing erroneous velocity values and applying smoothing (as described at the start of section B 3), data of reasonable quality have been derived for 12 events. Only 7 of these events are among the 31 discussed in the main report.

The maximum depth recorded at Worldsend was 419 mm in November 1996 but the corresponding flow was only 34 l/s due to blockages downstream - presumably at the culvert on the urban storm channel, which as seen in Plate B7d can contain a mix of trees and shopping trolleys. This backing up generally caused peak flows to be attenuated and peak depths to last for considerable periods. The flow/depth at Binfield lagged the *depth* at Worldsend by over 10 hours (see Fig. B16f) but was in advance of the maximum *flow* at Worldsend. The figure also shows the long duration of high depths at Worldsend.

The highest recorded flow, of 90 l/s (depth 140 mm), occurred in March 1995, and was unaffected by downstream blockages. The corresponding maximum flow at Binfield was 5.92 m^3/s and this lagged the Worldsend maximum flow by 50 minutes.

B3.10 Jealotts Ditch, Warfield (SU877720)

This gauge monitors runoff from a small pasture catchment (area 1.62 km^2) at a point where the drainage ditch (named Jealotts Ditch for the purposes of this report) crosses under a road (see Figs B4 and B6 marker K, and Plate B8). A Prolec Water-Rat was installed in February 1995 but a series of instrument problems meant satisfactory data were not obtained until October 1995. The instrument was damaged by a car careering into the ditch in December 1995, but generally good quality data were obtained from its reinstallation in February 1996 until the end of the field programme in March 1997. Data have been extracted for 9 events, but only 6 of these are among the 31 discussed in the main report. The data are reasonably 'noisy' but otherwise seem consistent and reliable, and minimal data processing was required.

The maximum depth recorded for Jealotts Ditch was 301 mm corresponding to a flow of 297 l/s. This occurred on 25 February 1996. The corresponding maximum flow at Binfield was 4.74 m^3/s which occurred over 5 hours earlier. Figure B16g shows the hydrographs at Jealotts Ditch and Binfield for this event and Fig. B16h shows the steady rating curve for Jealotts Ditch for the same event. It is interesting to note how the small rural catchment reacts more slowly and attenuates intense rainfall much more than the larger, partly urban, Binfield catchment. For certain summer storms no change in the flow through the pipe was detected (i.e. the pipe remained dry).

B3.11 Ascot Sewage Treatments Works (SU892683)

The outflow from Ascot STW (point L on Fig. B4) drains into the Bull Brook just downstream of the Warren Pond off-line flood storage, and upstream of the level gauge at Warfield House. Figures B5b & d show how the diurnal variation in Dry Weather Flow (DWF) from the works is clearly visible at both Warfield House and Binfield. It was therefore necessary to determine the average diurnal flow pattern (DWF profile) from the works and thus assess any increase in discharge due to storm runoff in Wet Weather conditions. A truly 'separate' sewage system would not show any effect, but in practice misconnections and infiltration into the sewer will always lead to some storm response.

Outflow from the works is measured daily at a rectangular thin plate weir (see Plate B8c). A downward seeking ultrasonic gauge (for automatic depth measurement) can be seen on Plate B8c, but this was not in working order. Thus, for this study a pressure transducer was installed upstream of the weir plate, and level data were collected at 15 minute intervals for a few months from September 1996 to February 1997. With the transducer set 51 mm below the weir crest, the following theoretical rating was derived to convert level h (m) to flow Q (m^3/s):

$$Q = 3.622\,(h-0.051)^{3/2}$$

The maximum level recorded was 0.213 m on 19 Nov 1996 at 11:30, during the 'storm period' of 16-24 November 1996. This level corresponded to a depth of water over the weir of 0.162 m and a flow of 0.236 m^3/s.

B 3.11.1 Raw level data analysis

Figure B17a shows each week of raw level data plotted against time from the 'start' of the week (taken as zero hours on Wednesday). The daily flow cycle is clearly seen, as are differences in profile between weekdays and weekends. The most obvious weekend/weekday difference is the 'shutdown' of flow at about 15:30 and 19:30 each weekday. This occurs due to an arrangement between Thames Water and the local electricity company, whereby on weekdays from November to January the works briefly shuts-down while it switches to using its own power generators over the period of peak (electricity) demand.

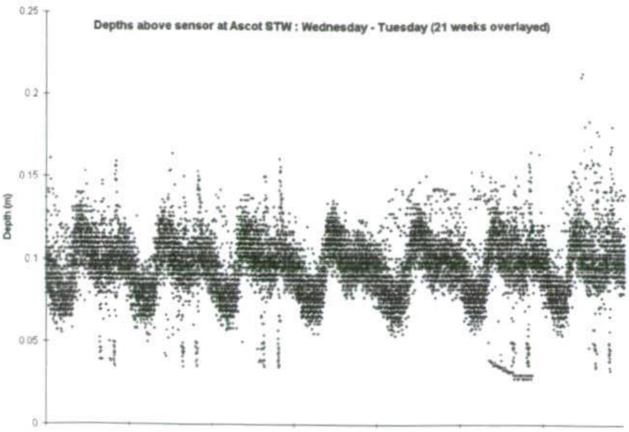
At each timestep during the week, minimum, maximum and mean values were derived, and plotted in Fig. B17b, showing a high degree of homogeneity and little spread. To clarify the apparent temporal differences between weekday and weekend profiles, mean profiles obtained for each day of the week were plotted over each other in Fig. B17c. The daily minimum depth (shut-down excluded) is seen to occur at 05:15, both during the week and at the weekend. Peak depths occur at around 09:00 during the week, between about 10:00 until 12:00 on Saturdays, and not until after midday on Sundays. On week-days, brief surges can be seen after shut-down periods.

B 3.11.2 Diurnal Flow Patterns

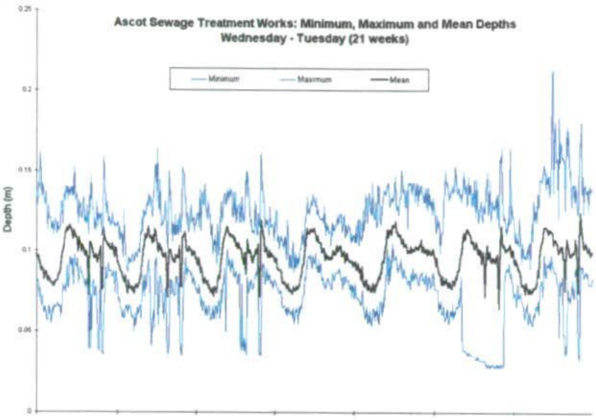
Using the derived rating equation, the raw level data were converted to flow rates, and average flow profiles derived for each day of the week. At each timestep during the week the average was calculated after trimming off the highest and lowest 10% of flow values in order to yield a more reliable DWF profile less influenced by high flows during storm periods. The average weekday profiles were then combined to give an overall weekday profile. Figure B17d shows the resulting DWF profiles, while Table B13 summarises the profile extremes and their time of occurrence.

Graphs for Ascot Sewage Treatment Works

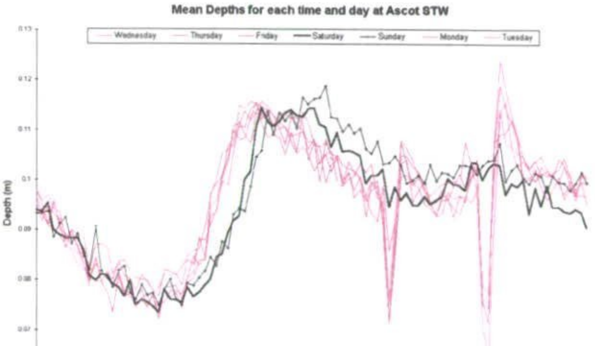
(a)



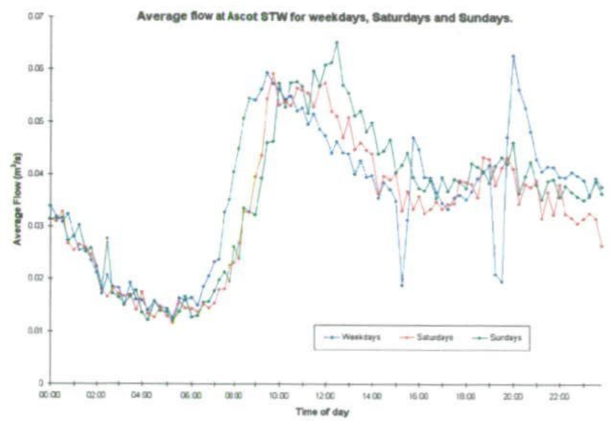
(b)



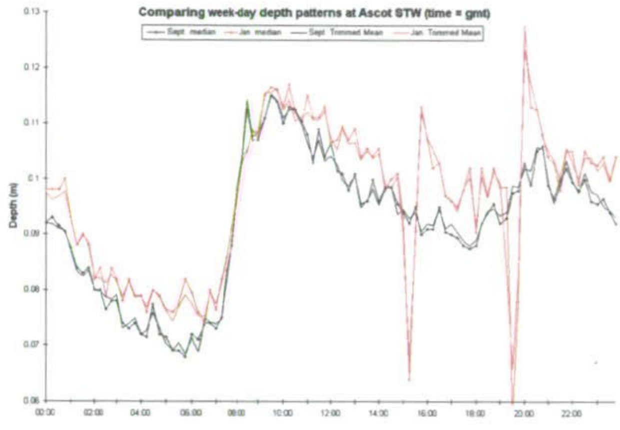
(c)



(d)



(e)



(f)

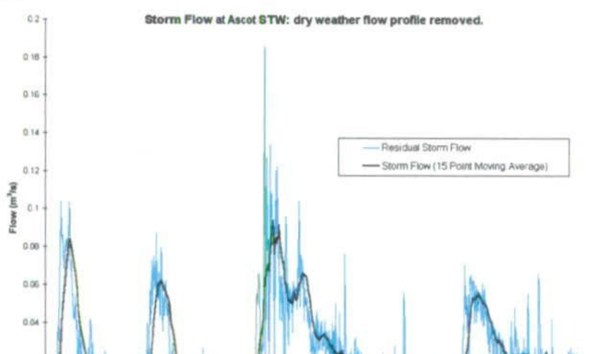


Figure B17

Table B13 Maxima and minima in average diurnal flow patterns at Ascot STW

		Minimum	Maximum	2^{nd} Maximum
Weekdays	Time	05:15	09:30	20:00
	Flow (m^3/s)	0.0120	0.0583	0.0589
Saturdays	Time	05:15	09:45	12:00
	Flow (m^3/s)	0.0116	0.0592	0.574
Sunday	Time	05:15	12:30	n/a
	Flow (m^3/s)	0.0122	0.0653	n/a

B 3.11.3 Examination of seasonal effects

To assess seasonal effects, weekday data from September were compared with weekday data from January (see Fig. B17e). The main difference between January and September is clearly the shut-down period - only seen during November to January. Otherwise, mean January flows are approximately $0.005m^3/s$ (or about 6%) greater than mean September flows. This could be due to groundwater infiltration increasing baseflow, or possibly changes in domestic use through the year. Similar results are found whether using median or trimmed mean averages, suggesting the increase is not due to increased rainfall during January.

B 3.11.4 Identifying Storm Flows

Having obtained the mean Dry Weather Flow pattern, it was possible to subtract this from the raw flow data, leaving the flow caused by storm events. The resulting flow series contained much 'noise' since the smoothing effect of averages was no longer present. However, a few storm periods were clearly visible on plots of residual flow against time. The four biggest storms, during the period data were collected at Ascot STW, all occurred during November. They are shown concatenated in Fig. B17f, with the low flow periods between them omitted. In practice, storm flow at this point in the Ascot catchment must be heavily damped, and a 15-point moving average could be applied in order to filter out the noise. Thus the peak of the unsmoothed data (plotted in blue) for 19 November 1996 was reduced from 0.187 m^3/s to 0.096 m^3/s. This is approximately 1½ times the peak Dry Weather Flow, and unsurprisingly, it occurred at the same time as the peak total flow. The volume of storm flow through the STW for this storm was approximately 7600 m^3.

Analysis of the data showed that peak storm flows were considerably larger than the DWF. Also, the DWF peak alone is higher than many of the storm peaks found at the gauges measuring runoff from small, rural, catchments (i.e. Worldsend, Jeallot's ditch). For example the peak flows on 19 November 1996 at Worldsend and Jeallot's were 34 l/s and 21 l/s respectively. In comparison with the peak flow at Warfield House of 5.0 m^3/s, however, the effect of Ascot STW was relatively minor. Thus flows from Ascot STW have not been included in the modelling work described in Chapter 5 of the main report.

B.4 OBSERVED FLOODING IN THE EVENT OF 12 OCTOBER 1993

Plates B9 to B11 show flooding observed at different locations in the catchment for a severe storm on 12 October 1993.

Plate B9 shows land drainage flooding at SU921711. There is no upstream urbanisation, but the manhole cover on the surface drain at the crossroads in (a) has been blown off. The flooding here may be exacerbated by raised levels in the downstream receiving water (the ornamental lake in Ascot Place).

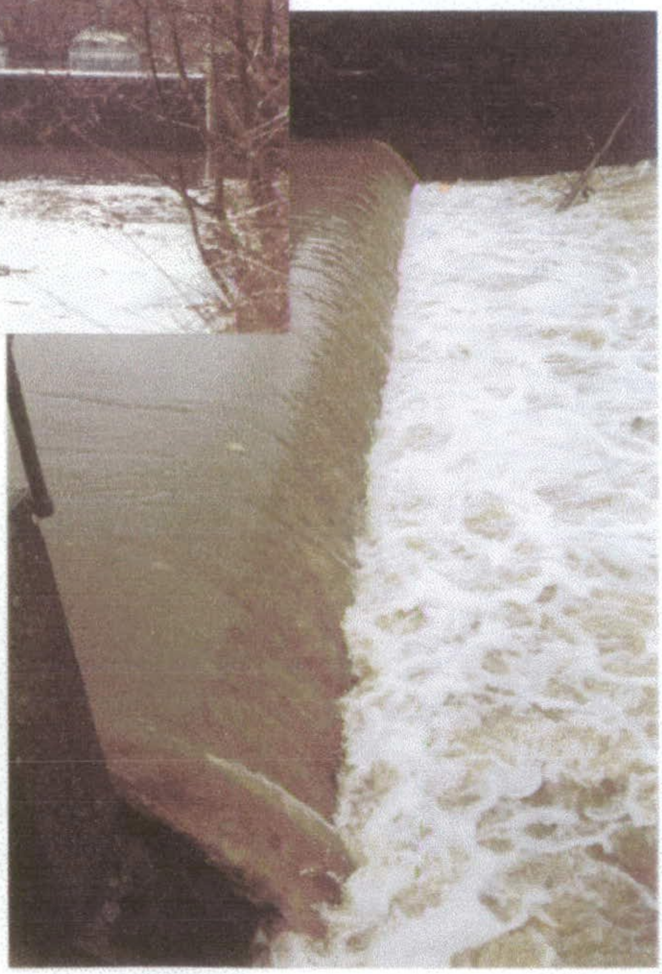
Plate B10a shows an abandoned car at SU892716, with the railings leading to Brockhill Bridge (see B 3.4) in the background. Plate B10b shows the flooded Montessori school adjacent to Wane Bridge, and Plate B10c shows the overfull Savemake storage pond.

Plate B11 shows in (a) a forest stream close to (but not at) Worldsend. The culvert under the road is blocked, forcing water to pond under the trees (b) until it can flow over the kerb onto the road (c). The flooding was generally not serious, but none was of urban cause or in the urban area.

B.5 PLATES

Cut at Binfield

a) Looking downstream
b) Looking upstream
c) Weir



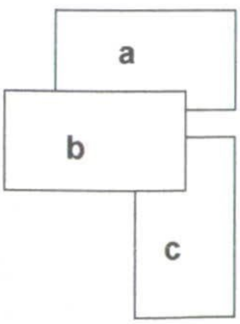


Plate B1

NRA/EA level recorder sites

a) Bull Brook at Warfield House looking dowstream
b) The Cut at Wane Bridge looking downstream. Flood of 12/10/93

a

b

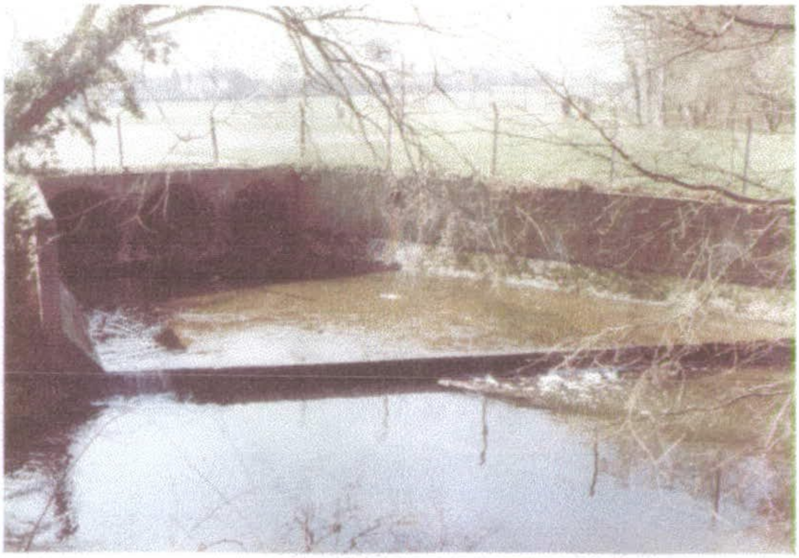
Plate B2

Views of Triple Bore Sewer at Jocks Lane

a) Cut enters from left

b) Cut leaves towards camera

c) Discharge apron under high flow conditions (middle pipe drains largest area)



a

b

c

Plate B3

Mill Pond

a) looking upstream from outlet

b) Inlet from Easthampsted/South Hill Park

a

b

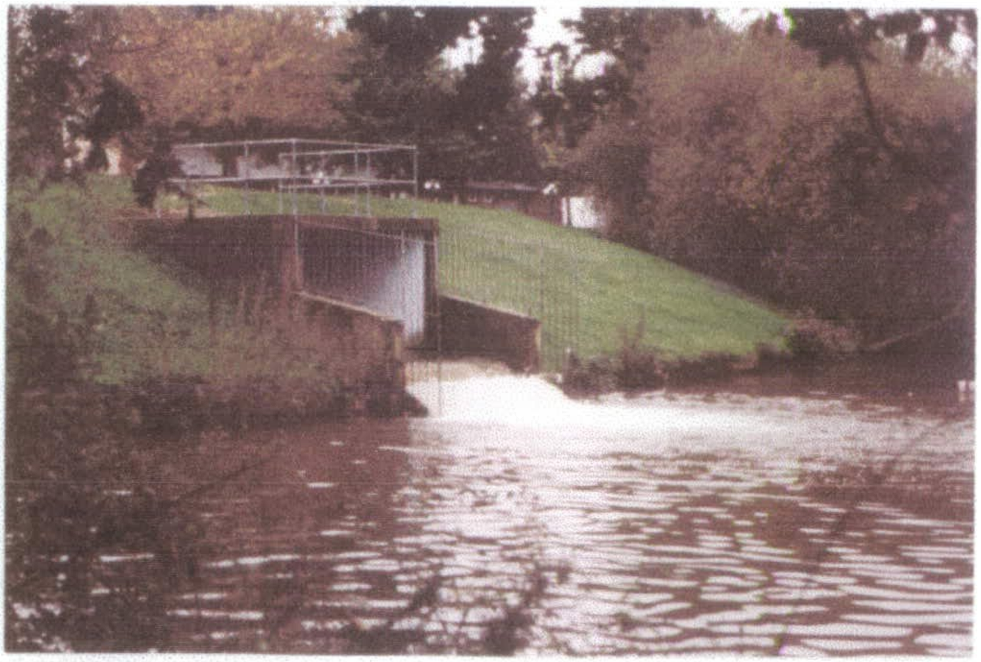


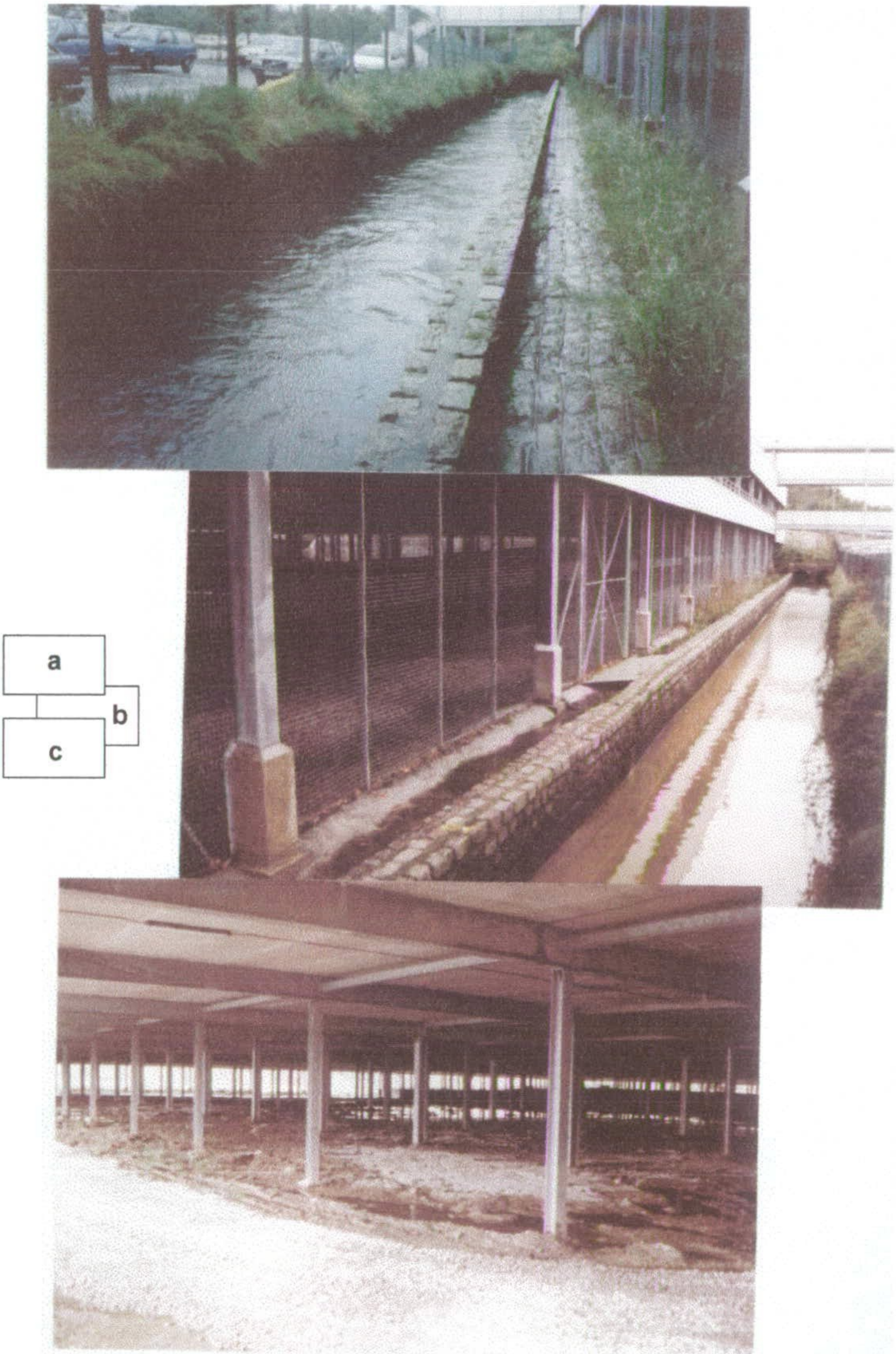
Plate B4

Oldbury Pond

a) Side weir + throttle pipe, looking downstream

b) Side weir + inlets, looking upstream

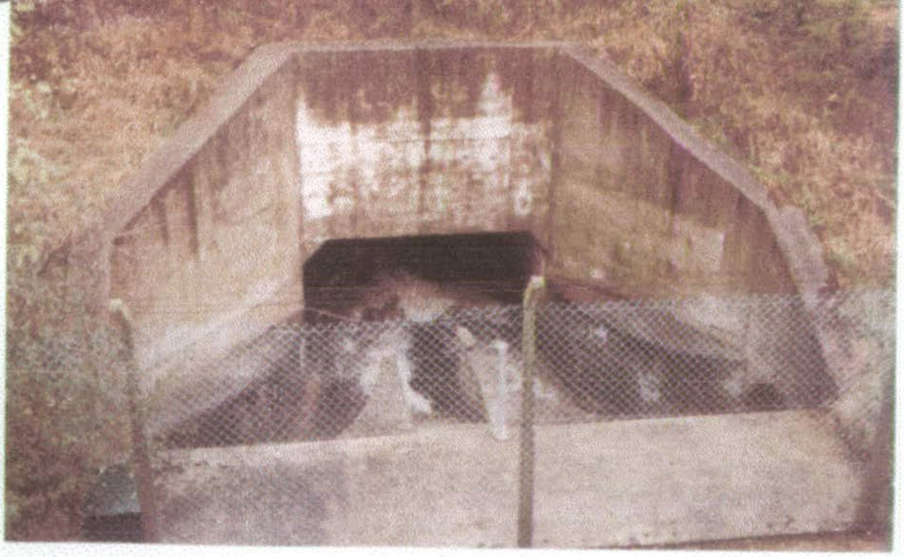
c) Off-line storage area

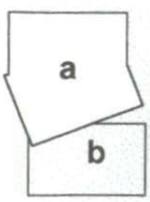


Oldbury Pond

a) Bypass channels at outlet

b) Outlet looking downstream





Ditch in forest at Worlds End

a) In forest
b) Obstructed, (free flowing) culvert under road
c) Outlet from obstructed culvert
d) Looking downstream, culvert (c) entering channel from right, shopping trolley obstructing next culvert

Plate B7

Jealotts Ditch and Ascot STW

a) Jealotts ditch culvert entrance

b) Jealotts ditch looking upstream

c) Ascot Sewage Treatment Works outflow weir

Plate B8

Event of 12/10/93
Land drainage flooding upstream of Ascot Lake

Plate B9

Event of 12/10/93

a) Abandoned car near Brockhill Bridge

b) Montessori School by Wane Bridge

c) Savernake flood storage pond

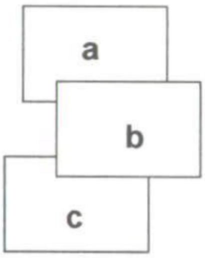


Plate B10

Event of 12/10/93

a) A forest ditch near Worldsend

b) Flooding just downstream from ditch

c) Flood water flowing over road

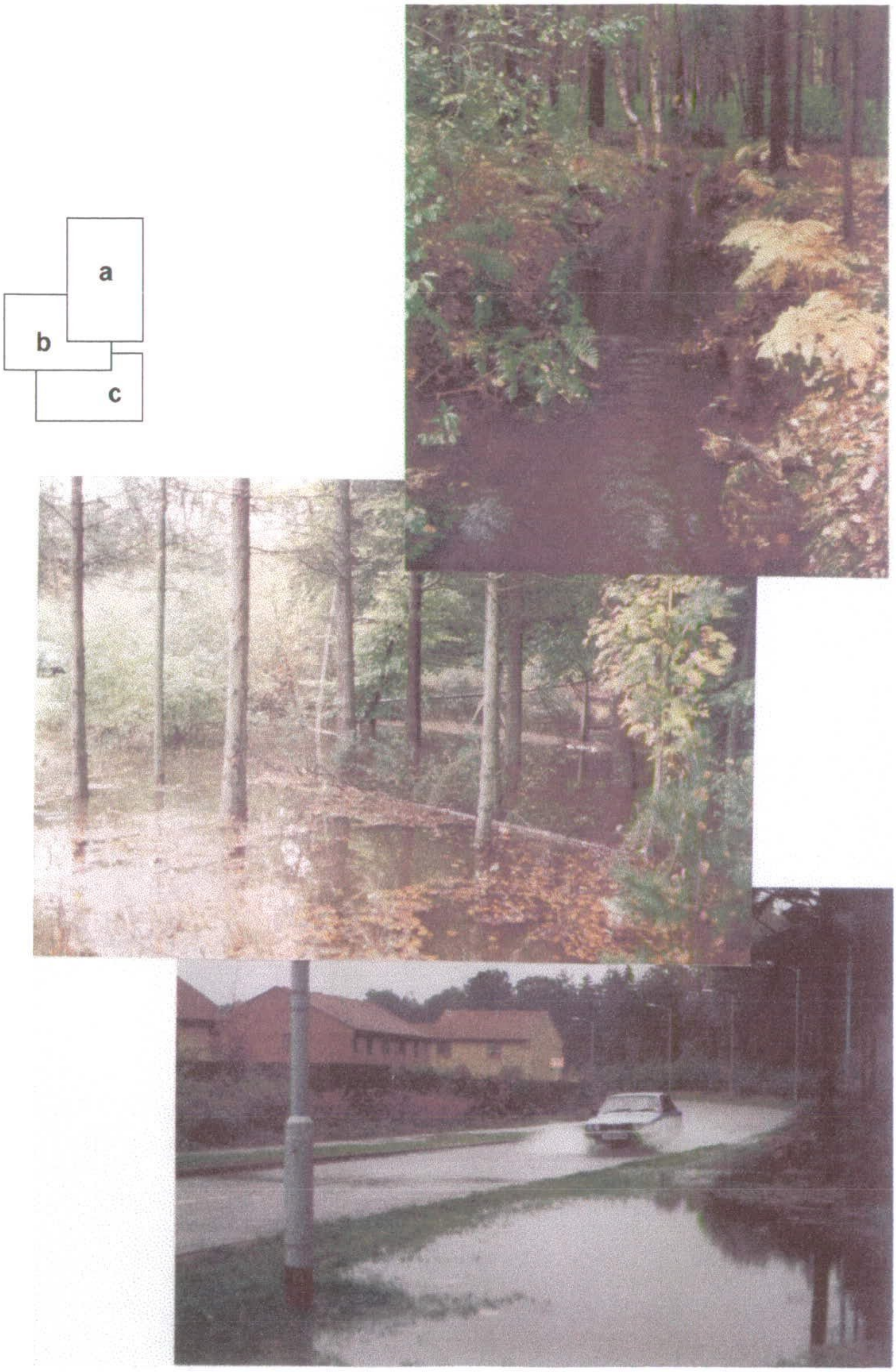


Plate B11